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PROCEEDINGS

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OF

CIVIL ENGINEERS

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OF THE

AMERICAN SOCIETY

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CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XLI—No. 7

SEPTEMBER, 1915

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1915

American Society of Civil Engineers

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To Codify Present Practice on the Bearing Value of Soils for Foundations, etc.: Robert A. Cummings, Edwin Duryea, Jr., E. G. Haines, Allen Hazen, James C. Meem, Walter J. Douglas.

On a National Water Law: F. H. Newell, George G. Anderson, Charles W. Comstock, Clemens Herschel, W. C. Hoad, Robert E. Horton, John H. Lewis, Charles D. Marx, Gardner S. Williams.

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The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

House of the Society—220 West Fifty-seventh Street, New York.

Telephone Number........................1446 Circle.
Cable Address........................"Ceas, New York."

* Vacancy in chairmanship caused by the death of Austin Lord Bowman.

Vol. XLI. SEPTEMBER, 1915. No. 7.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

SOCIETY AFFAIRS

CONTENTS

MINUTES OF MEETINGS

OF THE SOCIETY

September 1st, 1915.—The meeting was called to order at 8.30 P. M.; Director Arthur S. Tuttle in the chair; Chas. Warren Hunt, Secretary; and present, also, 123 members and 10 guests.

The minutes of the meetings of May 19th and June 2d, 1915, were approved as printed in the August, 1915, *Proceedings.*

A paper by H. R. Stanford, M. Am. Soc. C. E., entitled "Pearl Harbor Dry Dock" was presented by the author and illustrated with lantern slides.

The Secretary announced that he had received written communications on the subject from Messrs. F. R. Harris, Harrison S. Taft, C. E. Fowler, W. F. Frear, and P. L. Reed.

A paper by E. E. Howard, M. Am. Soc. C. E., entitled "The Twelfth Street Trafficway Viaduct, Kansas City, Missouri", was presented by the author and illustrated with lantern slides.

The paper was discussed by Messrs. M. M. Upson, and T. Kennard Thomson. The Secretary presented communications on the subject from Messrs. F. W. Green, L. J. Mensch, and H. W. Holmes, and the author replied briefly.

The Secretary presented the following report of the Tellers appointed to canvass the letter-ballot on the question:

"Shall the action of the Society on January 18th, 1911: 'That it is the sense of this meeting that the licensing of Engineers by States is undesirable' be rescinded?"

Total number of ballots received			2 879
Ballots from members in arrears of dues		230	
Ballots blank		8	
Ballots marked "?"		4	
Ballots unsigned		17	
Ballots stamped, not signed		4	263
Ballots counted			2 616
Number voting "Yes"	1 195		
" " "Yes" with remarks	29	2 024	
" " "No"	579		
" " "No" with remarks	13	592	2 616

Respectfully submitted,

CHAS. WARREN HUNT,
LINCOLN BUSH.

The chair declared that the action of the Society in this matter at the Annual Meeting of January 18th, 1911, had been rescinded.

At the request of J. H. Gandolfo, Assoc. M. Am. Soc. C. E., the Secretary presented a letter from him commenting on the wording of the letter-ballot relating to the licensing of engineers.

No action was taken.

The Secretary announced the election of the following candidates on August 31st, 1915:

As MEMBERS

RICHARD BEYER, Hoboken, N. J.
WILLIAM GEORGE BLIGH, Toronto, Ont., Canada
THOMAS HENRY CARVER, Seattle, Wash.
OLIVER HAMLINE DICKERSON, Duluth, Minn.
SAMUEL FORTIER, Washington, D. C.
CLINTON HALL KEARNY, San Antonio, Tex.

John Alexander McGrew, Albany, N. Y.
Joseph S Morrison, Ottumwa, Iowa
Louis Henry Prell, New Richmond, Ohio
Orville Campbell Skinner, Burnham, Pa.
George Pliny Smith, Cincinnati, Ohio
Jay L Stannard, Portland, Ore.

As Associate Members

Edwin Learned Adams, Los Angeles, Cal.
Edmund Oliver Archibald, Ship Creek, Alaska
Frank Arthur Banks, Moran, Wyo.
Archer Fortescue Barnard, Los Angeles, Cal.
Edward Vahan Baron, Yuma, Ariz.
Harold Ingersoll Bell, Portland, Me.
George Herbert Biles, Harrisburg, Pa.
James Alphonsus Boyle, Wilkes-Barre, Pa.
Walter Thompson Brooks, Kansas City, Mo.
Roy Wilson Burks, Louisville, Ky.
George Raymond Campbell, Spokane, Wash.
Charles Andrew Case, Joplin, Mo.
Gerald Otley Case, New York City
Herbert Augustine Claiborne, Jr., Richmond, Va.
Leland Clapper, Two Harbors, Minn.
Harry Lee Clarke, Mechanicville, N. Y.
Henry Edward Coane, Melbourne, Victoria, Australia
Richard Earl Cotton, Chicago, Ill.
John Francis Covert, San Diego, Cal.
Arthur Emmett Cowell, Merced, Cal.
George Solomon Crites, Tucson, Ariz.
Thomas Francis Curran, Utica, N. Y.
William James Davis, Winnipeg, Man., Canada
Arvin J Dillenbeck, Buffalo, N. Y.
Joseph Francis Sinnott Donnelly, Camagüey, Cuba
William Dyer, Mt. Vernon, Ohio
John Farris, Pittsburgh, Pa.
Ramiro Antonio Fernandez, New York City
George Fuchs, Tampa, Fla.
Fred Bacon Greenleaf, Auburn, Me.
Homer Huston Haggard, Havana, Cuba
Philip Jewett Hale, Chicago, Ill.
George Washington Hand, Chicago, Ill.
Luke Joseph Hayes, Niagara Falls, N. Y.
Walter Leo Hempelmann, St. Louis, Mo.
Harry Anderson Hickman, Vandalia, Ill.
George Leyburn Hughes, Norfolk, Va.

Andrew P Hustad, Minneapolis, Minn.
Andrew William Jackman, New Orleans, La.
Thomas McLean Jasper, London, England
Yeizo Kasano, Kobé, Japan
Earl Wallace Kelly, Duluth, Minn.
Edgar Augustus Kimmel, Lithia Springs, Ga.
Emanuel Carl Hjalmar Klingberg, Troy, N. Y.
William Franklin Krahl, Jr., Chicago, Ill.
Herbert John Kuelling, Milwaukee, Wis.
Karl Barclay Kumpe, Victoria, B. C., Canada
Albert Larsen, Millville, Mass.
Rudolph Wendell Phillips LeBaron, Chicago, Ill.
Charles Granville Lewis, San Francisco, Cal.
Daniel Collins Lipscomb, Denison, Tex.
John Lodge, Brooklyn, N. Y.
Oswald Lupinski, Wilkinsburg, Pa.
Robert MacMinn, Pittsburgh, Pa.
Rossiter Magers McCrone, Bangkok, Siam
Harry Fontaine McFarland, Jr., St. Louis, Mo.
Roger Keys McGee, Pittsburgh, Pa.
Ernest Martin Merrill, Beckley, W. Va.
Clifford Bennett Moore, Long Island City, N. Y.
Samuel Roy Morrow, Jefferson City, Mo.
Frank Tiebout Myers, Hattiesburg, Miss.
Arthur Theodore Nabstedt, Schenectady, N. Y.
George Alexander Noren, Beacon, N. Y.
Robert Preston Parker, Fort Smith, Ark.
Alois Phillip Poirot, Belleville, Ill.
John Frederick Poland, Cleveland, Ohio
John Ignatius Quinn, Payson, Utah
Jesse Steele Ritchey, Towanda, Pa.
James Wynbourne Routh, East Rochester, N. Y.
Charles Adrian Sawyer, Jr., Waban, Mass.
George Fuerle Schlesinger, Columbus, Ohio
Leon Monroe Schoonmaker, Flushing, N. Y.
Howard Daniel Severance, Monterey, Cal.
Walter Taylor Shultz, Baltimore, Md.
Arthur Philip Skaer, Buffalo, N. Y.
Norton Quincy Sloan, Dayton, Ohio
Lloyd Smoyer, New York City
Paul Bertram Spencer, West Haven, Conn.
John Stearns, Los Angeles, Cal.
John Bernard Stein, Brooklyn, N. Y.
James Cad Terry, Curityba, Parana, Brazil
Charles Melville Upham, Wilmington, Del.
Charles Joseph Weaver, Savannah, Ga.

Barclay White, Philadelphia, Pa.
George Philip Winn, Nashua, N. H.
Robert Lee Wood, El Monte, Cal.
Thomas Temple Wright, Chattanooga, Tenn.

As Associates

Edward Alfred Plath, Bartow, Fla.
John Milton Ritchie, Philadelphia, Pa.

As Juniors

John Edward Anderson, London, England
Ethelbert Bacon, New York City
Alfred Abraham Berkwitz, New York City
Harry Waldo Cole, Iroquois Falls, Ont., Canada
Holton Cook, Louisville, Ky.
George Burrett Davidson, Iroquois Falls, Ont., Canada
Charles Edmund DeLeuw, Chicago, Ill.
William Charles Diehl, Cali, Colombia
Charles William Doerr, Cleveland, Ohio
Benjamin Goodman, New York City
Wellesley Carl Harrington, Albany, N. Y.
George Alfred Helmstetter, Syracuse, N. Y.
Harold Fiske Holley, Williams, Cal.
William Dudley Hunt, Galveston, Tex.
Boyle Irwin, Philadelphia, Pa.
Lawrence Wilson Kinnear, New York City
Charles Kirschner, New York City
Harry Raymond Leach, Saginaw, Mich.
Raymond Marx, New York City
Albert Emerson Mellon, Tampa, Fla.
Charles Harold Munson, Berkeley, Cal.
James Carroll Murton, Toronto, Ont., Canada
Charles Franklin Reaney, Waterloo, Iowa
Eric Houghton Rhodes, Brisbane, Australia
William Ernest Robinson, Springfield, Mo.
Garland Livingstone Rounds, Walnut, Iowa
Henry Aylesbury Stringfellow, Rochester, N. Y.
George Edward Warren, Baltimore, Md.
Frank Ignatius Wheeler, Jr., Towson, Md.
Emmet Cheatham Wilson, Savannah, Ga.

The Secretary announced the transfer of the following candidates on August 31st, 1915:

From Junior to Associate Member

Norman Nathaniel Barber, Ottumwa, Iowa
Harry Montefiore Bergman, New York City

RALPH MCLANE BOWMAN, Indianapolis, Ind.
MEYER DAVIS, Pittsburgh, Pa.
FRANCIS BONNER FORBES, New York City
FREDERICK SHELTON FOULKROD, Pittsburgh, Pa.
SAMUEL GORDON, San Diego, Cal.
GUY ALEXANDER GRAHAM, New York City
ARTHUR BROOKS GREEN, Portland, Me.
ARNOLD CHARLES KELLERSBERGER, Houston Heights, Tex.
WALTER HARLAN LECKLITER, Corning, Iowa
HARRIE LANGDON MUCHEMORE, San Francisco, Cal.
LEON BENEDICT REYNOLDS, Kansas City, Mo.
ANDREW PEACH ROLLINS, Natalia, Tex.
JAMES ALFRED SILSBEE, Elmira, N. Y.
EDWARD HAZZARD WEST, Louisville, Ky.
HENRY BELCHER WHEATCROFT, JR., New York City
HAROLD IRA WOOD, Richmond, Cal.

FROM JUNIOR TO ASSOCIATE

CARL SWEETLAND REED, New York City

The Secretary announced the following deaths:

AUSTIN LORD BOWMAN, of New York City, elected Associate Member, September 7th, 1892; Member, December 1st, 1897; died June 3d, 1915.

DEXTER BRACKETT, of Boston, Mass., elected Member, June 6th, 1888; died August 26th, 1915.

LOOMIS EATON CHAPIN, of Canton, Ohio, elected Junior, December 3d, 1884; Associate Member, September 7th, 1892; Member, November 4th, 1896; died June 21st, 1915.

MENDES COHEN (*Past-President*), of Baltimore, Md., elected Member, December 4th, 1867; died August 13th, 1915.

HERBERT WHEELER COWAN, of Denver, Colo., elected Member, June 3d, 1908; died May 29th, 1915.

Sir SANDFORD FLEMING, of Ottawa, Ont., Canada, elected Member, September 18th, 1872; died July 22d, 1915.

JOHN CHARLES WILLIAM GRETH, of Pittsburgh, Pa., elected Member, March 4th, 1913; died August 7th, 1915.

WILLIAM MACKENZIE HUGHES, of Chicago, Ill., elected Member, June 2d, 1880; died June 25th, 1915.

CHARLES CLEMONS ROSE, of Scranton, Pa., elected Member, April 4th, 1888; died July 17th, 1915.

HIRAM EVERETT TERRY, of Flint, Mich., elected Member, June 6th, 1911; died March 8th, 1915.

CHARLES DOD WARD, of New York City, elected Member, March 3d, 1869; died July 30th, 1915.

Frank Robert Williamson, of Chicago, Ill., elected November 7th, 1906; died July 11th, 1915.

James Wilhelm Carpenter, of Cleveland, Ohio, elected Junior, February 28th, 1911; Associate Member, June 24th, 1914; died June 10th, 1915.

James Blaine Miller, of Washington, D. C., elected Associate Member, April 2d, 1913; died May 18th, 1915.

John Haines Warder, of Chicago, Ill., elected Associate, March 7th, 1888; died August 30th, 1915.

Harold Crosby Stevens, of Charlotte, N. C., elected Junior, December 31st, 1913; died May 23d, 1915.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

October 6th, 1915.—8.30 P. M.—This will be a regular business meeting. A paper by John Vipond Davies, M. Am. Soc. C. E., entitled "The Astoria Tunnel Under the East River for Gas Distribution in New York City", will be presented for discussion.

This paper was printed in *Proceedings* for August, 1915.

October 20th, 1915.—8.30 P. M.—At this meeting, a paper by F. zur Nedden, Esq., entitled "Induced Currents of Fluids", will be presented for discussion.

This paper was printed in *Proceedings* for August, 1915.

November 3d, 1915.—8.30 P. M.—A regular business meeting will be held, and a paper by Karl R. Kennison, Assoc. M. Am. Soc. C. E., entitled, "The Hydraulic Jump, in Open-Channel Flow at High Velocity", will be presented for discussion.

This paper is printed in this number of *Proceedings*.

November 17th, 1915.—8.30 P. M.—At this meeting a paper by Edwin Duryea, Jr., M. Am. Soc. C. E., and H. L. Haehl, Assoc. M. Am. Soc. C. E., entitled "A Study of the Depth of Annual Evaporation from Lake Conchos, Mexico", will be presented for discussion.

This paper is printed in this number of *Proceedings*.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In making a search it sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is small. This method is particularly useful when there are drawings or figures in the text, which would be very expensive to reproduce by hand.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings*.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at

6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M., every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, L. R. Hinman, 1400 West Colfax Ave., Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, at 12.30 P. M., at the Albany Hotel.

Visiting members are urged to attend the meetings and luncheons.

Atlanta Association

The Atlanta Association of Members of the American Society of Civil Engineers was organized on March 14th, 1912. The Association holds its meetings at the University Club, Atlanta, Ga.

At the meeting of the Association on January 9th, 1915, the following officers were elected for the ensuing year: President, Park A. Dallis; First Vice-President, B. M. Hall; Second Vice-President, P. H. Norcross; Secretary-Treasurer, T. B. Branch.

Baltimore Association

On May 6th, 1914, the Baltimore Association of Members of the American Society of Civil Engineers was organized, a Constitution adopted, and the following officers were elected: J. E. Greiner, President; Francis Lee Stuart, First Vice-President; L. H. Beach, Second Vice-President; Harry D. Williar, Jr., Secretary-Treasurer; and Messrs. H. D. Bush, B. T. Fendall, B. P. Harrison, Calvin W. Hendrick, Oscar F. Lackey, M. A. Long, and A. A. Thompson, Directors.

At its meeting of September 2d, 1914, the Board of Direction considered and approved the proposed Constitution of the Baltimore Association of Members of the American Society of Civil Engineers.

Cleveland Association

The proposed Constitution of the Cleveland Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on January 6th, 1915.

The following officers have been elected: President, Willard Beahan; Vice-President, Robert Hoffmann; Secretary-Treasurer, George H. Tinker.

Louisiana Association

At the meeting of the Louisiana Association of Members of the American Society of Civil Engineers (New Orleans, La.), on April 14th, 1915, the following officers were elected for the ensuing year: J. F. Coleman, President; W. B. Gregory and A. M. Shaw, Vice-Presidents; Ole K. Olsen, Treasurer; and E. H. Coleman, Secretary.

Northwestern Association

The proposed Constitution of the Northwestern Association of Members of the American Society of Civil Engineers (St. Paul and Minneapolis, Minn.) was considered and approved by the Board of Direction of the Society on November 4th, 1914. F. W. Cappelen is President and R. D. Thomas, Secretary.

Philadelphia Association

The meetings of the Philadelphia Association of Members of the American Society of Civil Engineers are held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

The officers of the Association are as follows: President, Richard L. Humphrey; Vice-Presidents, F. Herbert Snow and Edgar Marburg; Directors, John Sterling Deans, J. W. Ledoux, H. H. Quimby, and H. S. Smith; Treasurer, S. M. Swaab; and Secretary, W. L. Stevenson.

Portland, Ore., Association

At the meeting of the Association on October 21st, 1914, the following officers were elected for the ensuing year: President, George C. Mason; First Vice-President, W. S. Turner; Second Vice-President, John T. Whistler; Treasurer, G. B. Hegardt; and Secretary, Charles J. McGonigle.

St. Louis Association

The proposed Constitution of the St. Louis Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on October 7th, 1914.

The following officers have been elected: President, J. A. Ockerson; First Vice-President, Edward E. Wall; Second Vice-President, F. J. Jonah; Secretary-Treasurer, Gurdon G. Black. The meetings of the Association are held at the Engineers' Club Auditorium.

San Diego Association

The San Diego Association of Members of the American Society of Civil Engineers was organized on February 5th, 1915, and officers have been elected, as follows: President, George Butler; Vice-President, Willis J. Dean; and Secretary-Treasurer, J. R. Comly.

Seattle Association

The Seattle Association of Members of the American Society of Civil Engineers was organized on June 30th, 1913. At its meeting of January 25th, 1915, the following officers were elected for the ensuing year: President, R. H. Ober; Vice-President, A. S. Downey; and Secretary-Treasurer, Carl H. Reeves.

(Abstract of Minutes of Meetings)

July 26th, 1915.—The meeting was called to order at 12.15 P. M., at the College Club; A. S. Downey, Vice-President, in the chair; Carl H. Reeves, Secretary; and present, also, 29 members and guests.

The special order of business for the meeting being further discussion of the Articles of Association of The Associated Engineering Societies of Seattle, the Secretary read the Amendments presented by Mr. A. H. Fuller, at the meeting of June 28th, 1915. The discussion was opened by Mr. Fuller, who was followed by Messrs. Hedges, Hall, Gray, Downey, Howes, Dimock, and Tucker.

A vote on the Amendments, section by section, was called for, but a difference of opinion having developed as to whether affiliation such as that outlined by the Articles of Association was desired, a substitute motion was made by Mr. A. H. Dimock as follows:

"That this organization again endorses the principle of organizing the engineers of the City, and re-refers the matter to our Conference Committee to bring back a more specific arrangement with the Engineers' Club."

This motion, being duly seconded, was carried by a vote of 12 to 11, the deciding vote being cast by the Chair.

Mr. Hedges then moved "That it is the sense of this Association that we do not go into the Engineers' Club or any other amalgamation, except as each affiliated society goes into the amalgamation as a group". The motion was duly seconded.

On motion, it was decided to adjourn the meeting until August 2d, 1915, the hour and place to be arranged by the Secretary.

August 2d, 1915 (Adjourned Meeting).—The meeting was called to order at 8.00 P. M., in the New Seattle Chamber of Commerce; President R. H. Ober in the chair; Carl H. Reeves, Secretary; and present, also, 17 members and guests.

The minutes of the meetings of June 28th, and July 26th, 1915, respectively, were read and approved.

Mr. S. H. Hedges withdrew the motion *in re* amalgamation with other engineering societies of Seattle, offered by him at the meeting of July 26th, which was before that meeting at its adjournment.

Mr. B. D. Dean read an excerpt from an address by Charles Warren Hunt, Secretary of the Society, *in re* the co-operation of engineers, and supplemented the reading with a few remarks.

In accordance with the action of the Association at its meeting of July 26th, 1915, the Conference Committee presented a report containing certain changes and additions to the Articles of Association of The Associated Engineering Societies of Seattle, and the resolution of adoption was read by Robert Howes, Chairman of the Committee. The adoption of the Committee's report was moved by Mr. John L. Hall and seconded by Mr. Howes. The debate on the motion was opened by Mr. Hall, others discussing the subject being Messrs. Reeves, Herring, Hussey, Jacobs, Hedges, Fuller and Powell.

On motion by Mr. Joseph Jacobs, the following substitute was adopted:

"That this organization favor the formation of an association of engineering societies of Seattle, and that the sub-committee be enlarged

and authorized to recast the constitution submitted here this evening, changing it in so far as it refers to the special relationship with The Engineers' Club."

This motion was duly seconded, and the previous motion for the adoption of the report of the Conference Committee having been withdrawn, was carried.

President Ober appointed Messrs. Jacobs and Hedges as the additional members of the Conference Committee.

Adjourned.

Southern California Association

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bimonthly meetings, with banquet, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary of the Association, W. K. Barnard, 514 Central Building, Los Angeles, Cal.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

Spokane Association

The proposed Constitution of the Spokane Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on March 4th, 1914. Ulysses B. Hough is President.

Texas Association

The proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on December 31st, 1913. The headquarters of the Association is Dallas, Tex. John B. Hawley is President.

Special Committee on Materials for Road Construction

July 7th, 1915.—The meeting was held at the House of the Society. Present, W. W. Crosby (Chairman), H. K. Bishop, A. W. Dean, Nelson P. Lewis, Charles J. Tilden, George W. Tillson, and A. H. Blanchard (Secretary).

The minutes of the meeting of May 24th, 1915, were read and approved.

Mr. Lewis reported for the Sub-committee on Cost Data, and presented for consideration a form covering cost data for any type of road or pavement. The form was adopted as amended.

Mr. Tillson presented a progress report for the Sub-committee on Tests and Analyses.

It was decided to hold the next meeting of the Committee at the House of the Society at 9.30 A. M., on August 13th, 1915.

August 13th, 1915.—The meeting was held at the House of the Society. Present, W. W. Crosby (Chairman), H. K. Bishop, A. W. Dean, Nelson P. Lewis, Charles J. Tilden, George W. Tillson, and A. H. Blanchard (Secretary).

The minutes of the meeting of July 7th, 1915, were read and approved as amended.

Mr. Lewis, Chairman of the Sub-committee on Cost Data, presented a revised form covering cost data for any type of road or pavement under the following headings: foundation, wearing course, traffic data, and yearly cost per square yard. The form was discussed in detail, and certain amendments relative thereto were presented and adopted. On motion, duly seconded, the form was adopted and referred to the Sub-committee on Forms.

Mr. Crosby presented a tentative draft of the 1916 Report, and pages 1 to 4 were thoroughly discussed, amended, and tentatively approved.

On motion, duly seconded, the Chairman was directed to appoint sub-committees, each to consist of one member, to submit conclusions pertaining to the use of the following non-bituminous road materials: broken stone, broken slag, gravel, cement-concrete, paving brick, wood block, and stone block, and that each sub-committee forward its conclusions to the Secretary on or before September 15th, 1915.

The Chairman appointed the Sub-committees as follows: On Gravel, Mr. Bishop; on Broken Stone and Broken Slag, Mr. Dean; on Paving Brick, Mr. Lewis; on Cement-Concrete, Mr. Tilden; on Wood Block, Mr. Tillson; and on Stone Block, Mr. Blanchard.

Mr. Tillson, Chairman of the Sub-committee on Tests and Analyses, presented a report covering lists of tests to be made on each of the several non-bituminous highway materials, and methods for conducting tests. After discussion, the report was tentatively approved as amended.

It was decided to hold the next meeting of the Committee at the House of the Society at 9.00 A. M., on October 23d, 1915.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

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Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 176 Mansfield Street, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Detroit Engineering Society, 46 Grand River Avenue, West, Detroit, Mich.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, 6 West Eager Street, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Club of Trenton, Trent Theatre Building, 12 North Warren Street, Trenton, N. J.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, The Minories, Tower Hill, London, E., England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

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Louisiana Engineering Society, State Museum Building, Chartres and St. Ann Streets, New Orleans, La.

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Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Oregon Society of Civil Engineers, Portland, Ore.

Pacific Northwest Society of Engineers, 312 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

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Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From July 26th to September 1st, 1915)

DONATIONS*

MODERN BRITISH PERMANENT WAY:

Treating of Rails, Chairs, Fishbolts, Fishplates, Keys, Sleepers, Ballast, Rail-Joints, Points and Crossings, etc. By Cecil J. Allen. Cloth, 10½ x 8¼ in., illus., 17 + 147 pp. London, The Railway News, 1915. 6 shillings.

The subject-matter contained in this book was published, it is stated, as a series of articles in the *Railway News*, between 1911 and 1914. It has now been completely revised and brought up to date, and illustrations have been added. In it the author describes, it is said, in a comprehensive manner, the design of the various details which go to make up the completed track, together with the processes of their manufacture, as well as their possibilities and limitations. The book is intended, it is stated, for those engaged in permanent way drafting, who are not familiar with the details of design and manufacture. The Contents are: The History and Development of the Modern Bull-Headed Steel Rail; The Manufacture and Composition of Rail Steel; The Rolling, Testing and Inspection of Steel Rails; Fishplates; Fish-Bolts; The Design of Chairs; The Manufacture of Cast-Iron Chairs; Keys; Chair Fastenings; The Selection of Timber for Sleepers; The Preparation and Preservative Treatment of Sleepers; Ballast; Rail-Joints, Creep, and the Spacing of Sleepers; The Design of Switches; Examples of Switch Practice; Three-Throw Switches; Types of Crossings; The Design of Crossings; Examples of Crossing Practice; Slip Roads and Scissors Crossings; Manganese Steel Switches and Crossings; Appendix: Revised British Standard Specification for Fishplates, December, 1914.

ELECTRIC ELEVATORS: THEIR CONSTRUCTION AND OPERATION.

By Elmer G. Henderson. Paper, 7½ x 5 in., illus., 90 pp. Chicago, The Branch Publishing Company, 1915. $1.00.

The subject-matter of this book, it is stated, is devoted to detailed descriptions, with numerous illustrations, of the construction and operation of the various types of electric elevators for passenger and freight service, as well as of the kinds of motors to be used, the controllers, governors, etc. The author has included in his text the Chicago building ordinance of 1913 regulating the construction and maintenance of passenger and freight elevators, and, for ready reference, has given fourteen tables which contain data as to motor currents, sizes of wire, fuses, etc., for use in the various types of installations. The Contents are: Parts and Classes of Elevators; Direct Connected Electric Elevators; Counterbalancing of Electric Elevators; Direct Current Elevator Motors; Alternating Current Elevator Motors; Direct and Alternating Current Elevator Controllers; Design and Operation of a Modern Controller; Construction and Operation of Controllers; Governors; Automatic or Push-Button Elevators; Building Ordinances; Useful Tables; Index.

THE NAVAL POCKET-BOOK, 1915.

Founded by *Sir* W. Laird Clowes. Edited by R. C. Anderson. Twentieth Year. Cloth, 5¼ x 3¾ in., illus., 29 + 716 pp. London, W. Thacker & Co.; Calcutta and Simla, Thacker, Spink & Co., 1915. 7 shillings 6 pence.

As stated in the title, this volume is a pocket-book of the navies of the world corrected to April 7th, 1915. After preliminary explanatory matter, such as a method of classification to be applied to each fleet described, notes and abbreviations used in the tables, a summary of the fighting fleets of the world, etc., descriptions are given of the naval vessels of the various nations, arranged alphabetically by country, including class of vessel, names, place and date of laying down and launching, dimensions, displacement, power, types of engines, kind of armor used, armament, trial speeds, etc. There are also descriptions of the various types of naval guns manufactured by different firms, a list of dry docks in Europe, Asia, Africa, America, Oceanica and Australasia, including type, dimensions, etc., conversion tables of measures, plans of battleships and armored cruisers of the navies of the different nations, arranged alphabetically by country, and an index of names of ships mentioned in the text.

* Unless otherwise specified, books in this list have been donated by the publishers.

POCKET COMPANION FOR THE USE OF ENGINEERS, ARCHITECTS AND BUILDERS, 1915

Containing Useful Information and Tables Pertaining to the Use of Steel Manufactured by Dorman, Long & Co., Limited, Middlesbrough, England. Computed and Edited by the Constructional Department. Leather, 6½ x 4½ in., illus., 40 + 273 pp. Middlesbrough, William Appleyard and Sons, Limited, 1915. (Donated by Dorman, Long & Co., Limited.)

This book, the preface states, contains a full description of the products of each department of the Dorman, Long & Co., Limited, properties in various places, consisting of steel furnaces, rolling mills, structural and bridge shops, wire and wire rod mills, and rail, billet, bloom and slab mills, etc. The subject-matter, it is said, has been prepared and arranged for convenient reference in accordance with the best modern practice, to meet the requirements of engineers, architects, and those engaged in structural engineering, shipbuilding, and allied trades. The text is illustrated and contains tables giving dimensions, properties, etc., of the products described, and it is hoped that the book will be found useful by those requiring material manufactured by the company or engaged in designing structures for which its products are adapted. The Contents are: Notes on Sections; Dimensions, Properties, etc., of Sections; Information Relating to the Carrying Capacity, etc., of Beams, Compound Girders, Stanchions, Struts, Roof Trusses, Plate Girders and Troughing, with Standard Details; Sheet Department: Steel and Iron Sheets, Corrugated, Curved and Plain; Wire and Rod Department: Rolling Mills, Wire Drawing and Galvanizing Shops; Open-Hearth Steel: Specialities; Rolled Sections; General Information, Formulas, Tables, etc.; Index.

OXY-ACETYLENE WELDING AND CUTTING

Including the Operation and Care of Acetylene Generating Plants and the Oxygen Process for Removal of Carbon. By Calvin F. Swingle. Cloth, 6¾ x 4½ in., illus., 190 pp. Chicago, Frederick J. Drake & Co., 1915. $1.50.

Practically all metals, the preface states, are now welded by the oxy-acetylene flame, and manufacturers, contractors, etc., are said to be adding the process to their equipment because of its cheapness and speed over old methods. The author, in this book, has endeavored, it is stated, to cover every practical point in the process of welding and cutting and the removal of carbon with oxygen. The principal kinds of welding, such as fire welding, welding by water gas, thermit welding, brazing, and blowpipe welding, are described briefly, and their advantages and disadvantages are discussed. Sufficient theory is included in the subject-matter, it is said, to enable the practical man to acquire a thorough understanding of the subject, as well as numerous illustrations of distinct advantage to the operator. The Chapter headings are: Welding; Welding Flames; Oxygen; Acetylene; Acetylene Gas Purification and Handling; Oxy-Acetylene Torches; Characteristics of Welding Torches; Welding Installations; Preheating and Annealing; Operating a Welding Installation; Metal Welding Practice; Oxy-Acetylene Cutting; Oxygen Carbon Removal; Index.

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BY PURCHASE

Manual of Agricultural Chemistry. By Herbert Ingle. Third Edition. New York and London, 1913.

Lathes: Their Construction and Operation. By George W. Burley. New York and London, 1915.

Masonry as Applied to Civil Engineering. Being a Practical Treatise on the Design and Construction of Engineering Works in Stone and Heavy Concrete. By F. Noel Taylor. New York, 1915.

A Study of the Circular-Arc Bow-Girder. By A. H. Gibson and E. G. Ritchie. New York, 1915.

Single-Phase Electric Railways. By Edwin Austin. New York, 1915.

SUMMARY OF ACCESSIONS

(From July 26th to September 1st, 1915)

Donations (including 15 duplicates)	367
By purchase	5
Total	372

MEMBERSHIP

(From August 6th to September 2d, 1915)

ADDITIONS

MEMBERS

		Date of Membership.
CROWE, FRANCIS TRENHOLM. Constr. Engr., Boise Project, U. S. Reclamation Service, Boise, Idaho	Assoc. M. M.	May 3, 1910 July 7, 1915
ELROD, HENRY EXALL. Cons. Engr., Southwestern Life Bldg., Dallas, Tex	Assoc. M. M.	Nov. 7, 1906 July 7, 1915

ASSOCIATE MEMBERS

BERGMAN, HARRY MONTEFIORE. Supt., Godwin Constr. Co., 251 Fourth Ave. (Res., 615 West 143d St.), New York City	Jun. Assoc. M.	May 4, 1909 Aug. 31, 1915
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GLENN, RUSSELL VERSTILLE. Capitol City Club, Atlanta, Ga		June 3, 1915
GOODMAN, CHARLES. Asst. Engr., Board of Water Supply, City of New York, 10 Beach 43d St., Edgemere, N. Y		June 3, 1915
GRODSKE, WALTER JOHN. Asst. Engr., Bureau of Public Works, Manila, Philippine Islands		Oct. 7, 1914
JACOB, CLARENCE CECIL. Dist. Engr., Water Resources Branch, U. S. Geological Survey, 417 Fleming Bldg., Phœnix, Ariz		April 7, 1915
NORRIS, JOHN ALEXANDER. Wharton, Tex		Mar. 2, 1915
ROBSON, RALPH EWART. Res. Engr., Colony Holding Corporation, Atascadero, Cal	Jun. Assoc. M.	Oct. 3, 1911 June 3, 1915
STACK, JAMES RAYMOND. Engr. and Contr. (Stack Constr. Co.), 306 Sellwood Bldg., Duluth, Minn		April 7, 1915
VROMAN, GUY. Asst. Designing Engr., Board of Water Supply, City of New York, Larchmont, N. Y		Mar. 2, 1915

JUNIORS

DREYFUS, SAMUEL CELLNER. Box 10, Tavares, Fla	June 3, 1915
HEINONEN, HENRY JALMAR. 154 Halsey St., Brooklyn, N. Y.	April 7, 1915
LATHROP, HENRY MONROE. Eng. Draftsman, C. R. R. of N. J., 143 Liberty St. (Res., 39 Charles St.), New York City	June 3, 1915
LUCCHESI, DAVID HOWE, JR. Draftsman, Baltimore Sewerage Comm., 2208 McCulloh St., Baltimore, Md	April 7, 1915

CHANGES OF ADDRESS

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Abbot, Frederic Vaughan. Col., Corps of Engrs., U. S. A., U. S. Engr. Office, Army Bldg., Room 707, New York City.

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Hodges, Harry Foote. Brig. Gen., U. S. A., Fort Totten, N. Y.

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LeBaron, John Francis. Economic Geologist, R. F. D. 1, Box 118, Essex, Mass.

Love, Andrew Cavitt. Prof. of Railway Eng., Agricultural and Mech. Coll. of Texas, College Station, Tex.

McKim, James Arthur. 545 South Norton Ave., Los Angeles, Cal.

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ASSOCIATE MEMBERS

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BALDWIN, FRANCIS NEAL. Asst. Engr., Tex. & Pac. Ry., 218 Thalia St., New Orleans, La.

BECK, ALFRED FORTIN. Care, Am. Bridge Co., Gary, Ind.

BLIEM, DANIEL WILLIAM. Chf. Engr., Milliken Bros. (Inc.), 17 Battery Pl., New York City.

BODYCOMB, WALTER CHRISTMAS. With Westinghouse, Church, Kerr & Co., 37 Wall St., New York City.

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YEREANCE, ALEXANDER WOODWARD. Care, Union Powder Corporation, Parlin, N. J.

DEATHS

BRACKETT, DEXTER. Elected Member, June 6th, 1888; died August 26th, 1915.

COHEN, MENDES (*Past-President*). Elected Member, December 4th, 1867; died August 13th, 1915.

GRETH, JOHN CHARLES WILLIAM. Elected Member, March 4th, 1913; died August 7th, 1915.

TERRY, HIRAM EVERETT. Elected Member, June 6th, 1911; died March 8th, 1915.

Total Membership of the Society, September 2d, 1915, 7 793.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(July 26th to September 1st, 1915)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., St. Louis, Mo., 30c.
(2) *Proceedings*, Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur*, München, Germany.
(8) *Stevens Institute Indicator*, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, 432 Fourth Ave., New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *Engineering Record*, New York City, 10c.
(15) *Railway Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway Review*, Chicago, Ill., 15c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 20c.
(21) *Railway Engineer*, London, England, 1s. 2d.
(22) *Iron and Coal Trades Review*, London, England, 6d.
(23) *Railway Gazette*, London, England, 6d.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *Railway Age Gazette*, Mechanical Edition, New York City, 20c.
(26) *Electrical Review*, London, England, 4d.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *Cornell Civil Engineer*, Ithaca, N. Y.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Technisches Gemeindeblatt*, Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung*, Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift*, Berlin, Germany.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Colliery Engineer*, Scranton, Pa., 25c.
(46) *Scientific American*, New York City, 15c.
(47) *Mechanical Engineer*, Manchester, England, 3d.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia, 25 kop.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria, 70h.
(54) *Transactions*, Am. Soc. C. E., New York City, $12.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $6.

(57) *Colliery Guardian,* London, England, 5d.
(58) *Proceedings,* Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings,* American Water-Work Assoc., Troy, N. Y.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Steel and Iron,* Thaw Bldg., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 5c.
(65) *Official Proceedings,* New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting,* London, England, 6d.
(67) *Cement and Engineering News,* Chicago, Ill., 25c.
(68) *Mining Journal,* London, England, 6d.
(69) *Der Eisenbau,* Leipzig, Germany.
(71) *Journal,* Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs,* Iron and Steel Inst., London, England.
(72) *American Machinist,* New York City, 15c.
(73) *Electrician,* London, England, 18c.
(74) *Transactions,* Inst. of Min. and Metal., London, England.
(75) *Proceedings,* Inst. of Mech. Engrs., London, England.
(76) *Brick,* Chicago, Ill., 20c.
(77) *Journal,* Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen,* Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten,* Vienna, Austria.
(80) *Tonindustrie Zeitung,* Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen,* Wiesbaden, Germany.
(82) *Mining and Engineering World,* Chicago, Ill., 10c.
(83) *Gas Age,* New York City, 15c.
(84) *Le Ciment,* Paris, France.
(85) *Proceedings,* Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting,* Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way,* Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings,* Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions,* Inst. of Naval Archts., London, England.
(91) *Transactions,* Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin,* Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie,* Paris, France, 4 fr. 50.
(95) *International Marine Engineering,* New York City, 20c.
(96) *Canadian Engineer,* Toronto, Ont., Canada, 10c.
(98) *Journal,* Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings,* Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs,* Corps of Engrs., U. S. A., Washington, D. C., 50c.
(101) *Metal Worker,* New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens,* Wiesbaden, Germany.
(103) *Mining and Scientific Press,* San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer,* London, England, 6d.
(105) *Metallurgical and Chemical Engineering,* New York City, 25c.
(106) *Transactions,* Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung,* Zürich, Switzerland.
(108) *Iron Tradesman,* Atlanta, Ga., 10c.
(109) *Journal,* Boston Soc. C. E., Boston, Mass., 50c.
(110) *Journal,* Am. Concrete Inst., Philadelphia, Pa., 50c.
(111) *Journal of Electricity, Power and Gas,* San Francisco, Cal., 25c.
(112) *Internationale Zeitschrift für Wasser-Versorgung,* Leipzig, Germany.
(113) *Proceedings,* Am. Wood Preservers' Assoc., Baltimore, Md.

LIST OF ARTICLES

Bridges.

New-Old Theory of Reinforced Concrete Beams in Bending.* L. J. Mensch. (110) Dec., 1914.
Report of Committee on Reinforced Concrete Highway Bridges and Culverts. (Am. Concrete Inst.) (110) Feb.
Tests of Beam Connections.* C. S. Whitney. (36) May.
The Franklin Avenue-Louisiana Street Bridge for the City of Houston, Texas.* J. L. Jacobs. (36) May.
Wood Block and Granite for Bridge Floors.* Edward A. Byrne. (Paper read before the Am. Road Builders' Assoc.) (60) June.
Ohio River Bridge for the C., B. & Q. R. R.* (13) July 29.
Canal Bridges.* E. A. Cross. (11) Serial beginning July 30.
Surfacing Bridges.* Frank R. Lander. (60) Aug.
A Long Span Deck Girder Bascule Bridge.* (87) Aug.

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Bridges—(Continued).

Reconstruction of Piers of Little Rock Junction Bridge Across the Arkansas River at Little Rock, Ark.* (86) Aug. 4.
The Chesapeake & Ohio Northern Railway Bridge Over the Ohio River at Sciotoville, Ohio.* (86) Aug. 4.
Reconstruction of Mississippi River Bridge at Keokuk.* (13) Aug. 5.
An Ornamental and Cheap Concrete Bridge.* R. C. Hardman. (13) Aug. 5.
Canadian Pacific Draw Span Over the Lachine Canal.* (15) Aug. 6.
Reinforced-Concrete Arch Bridge has Stone Facing.* (14) Aug. 7.
Diagrams Facilitate Estimating Highway Bridges.* G. F. Burch. (14) Aug. 7.
Allowing for Impact in Bridge Calculations.* J. D. W. Ball. (12) Aug. 13.
A New Bridge Over the Missouri River at Kansas City.* (15) Aug. 13.
Solid Deck Trestles and Bridges on the Illinois Central.* (15) Aug. 13.
Direct-Lift Span Provides 55-Foot Clearance over Louisville and Portland Canal.* (14) Aug. 14.
A Simple Method for Determining Two-Hinged Arch Reactions.* C. S. Whitney. (86) Aug. 18.
Erection of Steel Arch Span, Detroit-Superior Viaduct.* (13) Aug. 19.
Recent Cylinder Pier Construction.* J. E. Bebb. (15) Aug. 20.
Renewing a Busy Main Line Bridge on the Santa Fe.* L. C. Lawton. (15) Aug. 20.
The Chattanooga Creek Bridge of the N. C. & St. L.* C. H. Johnson. (15) Aug. 20.
Erection at Hell Gate Arch Checks Calculations.* (14) Aug. 21.
Paving Problems of Queensboro Bridge, New York.* (13) Aug. 26.
Building Concrete Caissons in the Platte River.* J. H. Merriam. (15) Aug. 27.
Unit-Construction System Applied to a Three-Mile Concrete Viaduct to Reduce the Cost.* (14) Aug. 28.
Elastic Curve Applied to the Design of the Sciotoville Bridge.* D. B. Steinman. (14) Aug. 28.
Le Viaduc en Béton de Tunkhannock du Lackawanna Railroad, à Nicholson (Pennsylvania, E.-U.).* P. Calfas. (33) Aug. 7.
Calcul et Expérimentation des Poutres Solidaires de leurs Supports.* P. Caufourier. (33) Aug. 7.
Neues Verfahren zur raschen Ermittelung der Abmessungen und Eiseneinlagen von Gewölbefugen.* R. Färber. (51) Sup. No. 13, 1915.
Eisenbahnbrücke über die Saar bei Völklingen mit anschliessender Hochbahn.* H. Dürr. (51) Serial beginning Sup. No. 14, 1915.

Electrical.

The Relative Migration Velocities of the Ions in Complex Electrolytes. A. Mutscheller. (105) July.
Predetermination of the Residual Flux in Magnets.* Kenelm Edgcumbe. (73) July 16.
Resistance of Carbon Contacts in the Solid Back Transmitter.* A. L. Clark. (Abstract from *Physical Review.*) (73) July 16.
Units from Refuse. A. J. Abraham. (26) July 16.
The Central Power Station of the Randfontein Estates.* R. Turnbull Mawdesley. (26) July 16.
A Comparison of Different Methods of Testing Electrical Porcelain.* A. Chernyshoff and C. A. Butman. (Abstract from the *Electric Journal.*) (73) July 16.
Electric Control-Gear for Air-Compressor Motor.* (11) July 16.
The Electric Crane Applied to the Handling of Coal and Ore.* H. H. Broughton. (73) Serial beginning July 23.
The Weaverham and District Electricity Supply.* (26) July 23.
On an Unbroken Alternating Current for Cable Telegraphy.* George O. Squier. (Paper read before the Physical Soc. of London.) (26) Serial beginning July 30.
A 200-Mile Artificial Transmission Line. C. E. Magnusson, J. Gooderham and R. Rader. (From the *Electrical World.*) (73) July 30.
The Measurement of Self-Induction, an Alternate-Current Bridge.* D. Owen. (73) July 30.
A Short Method for Calculating Starting Resistance.* B. W. Jones. (From the *General Electric Review.*) (73) July 30.
Mobile Color and Stage Lighting.* Bassett Jones. (27) Serial beginning July 31.
Virginia Power Company's Cabin Creek Plant.* (27) July 31.
The Effect of Transient Voltages on Dielectrics.* F. W. Peek, Jr. (42) Aug.
Modern Theories of Magnetism.* George Flowers Stradling. (3) Aug.
Overhead Electrolysis and Porcelain Strain Insulators.* S. L. Foster. (42) Aug.
How Bell Invented the Telephone. Thomas A. Watson. (42) Aug.
Delta-Cross Connections of Transformers for Parallel Operation of Two and Three-Phase Systems.* George P. Roux. (42) Aug.

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Experimental Researches on Skin Effect in Conductors.* A. E. Kennelly, F. A. Laws and P. H. Pierce. (42) Aug.
Abnormal Voltages in Transformers. J. Murray Weed. (42) Aug.
Harmonics in Transformer Magnetizing Currents. J. F. Peters. (42) Aug.
The Trend of Electrical Development. Paul M. Lincoln. (42) Aug.
Physical Limitations in D. C. Commutating Machinery. B. G. Lamme. (42) Aug.
The Automatic Switchboard Telephone System of Los Angeles, Cal.* W. Lee Campbell. (42) Aug.
A Large Electric Hoist.* Wilfred Sykes. (42) Aug.
New Carquinez Straits High-Tension Crossing.* (13) Aug. 5.
The Electrical Design of Induction Motors.* H. L. Smith. (Paper read before the Rugby Eng. Soc.) (73) Serial beginning Aug. 6.
Coupled Oscillatory Circuits and the System à Onde Unique. G. W. O. Howe. (73) Serial beginning Aug. 6.
Insulating Properties of Solid Dielectrics.* Harvey L. Curtis. (Abstract from *Bulletin*, Bureau of Standards.) (73) Serial beginning Aug. 6.
Electrically Driven Sawmills.* (111) Aug. 7.
Wireless Call Devices.* L. B. Turner. (Abstract of paper read before the Institution of Post Office Elec. Engrs.) (73) Aug. 13.
Some Points in Transformers.* W. E. Burnand. (73) Aug. 13.
Sag-Tension Calculations. Harold Pender. (27) Aug. 14.
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Motors Simplify Operations in Milk Depot.* J. L. Wiltse. (27) Aug. 14.
Construction of a Vibrating Rectifier.* Charles Fraasa. (19) Aug. 14.
Current-Limiting Reactors.* W. H. Dann and H. H. Rudd. (Abstract of paper read before the Ohio Soc. of Mech., Elec. and Steam Engrs.) (64) Aug. 17.
Havana Consolidated Power Plant.* C. W. Ricker. (64) Aug. 17.
Saving $1 500 a Month by Using Electric Pumps.* E. E. Yensel. (27) Aug. 21.
Central-Station Rate Making. Paul J. Kiefer. (64) Aug. 24.
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Electrical Plant of the Wakefield Iron Co.* Holman I. Pearl and Joe Green. (16) Aug. 28.
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Continued Gain in Light and Power Industry.* (27) Aug. 28.
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Power-Station Economics. E. J. Billings. (Paper read before the Missouri Public Utilities Comm.) (64) Aug. 31.
The Delray Power Plants.* Norman G. Reinicker. (64) Aug. 31.
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Ueberspannungen mit der Betriebsfrequenz bei Leitungsbrüchen und einpoligen Schaltvorgängen.* W. Petersen. (41) Serial beginning July 16.
Die physiologischen Wirkungen elektrischer Starkströme bei Unfällen sowie die heutigen Wiederbelebungsmethoden und ihre Aussicht auf Erfolg.* K. Alvensleben. (41) Serial beginning July 29.
Spannungswellen und Stromwellen in Hochspannungsprüfanlagen.* Gustave Benischke. (41) Aug. 5.

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An Ice-Breaking Train Ferry Steamer. (12) July 16.
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Les Sous-Marins Allemands, leur Rôle dans la Guerre Actuelle. M. Laubeuf. (32) Jan.
La Mise à Feu par l'Electricité des Mines sous-Marines et des Fourneaux de Mines.* J. Vichniak. (33) July 24.
Die Form des Schraubenstrahles und seine Energieänderung.* H. Krey. (48) July 24.
Die Elektrizität an Bord von Schiffen.* O. Krell. (41) Serial beginning Aug. 12.

Mechanical.

The Annealing of Cold-Rolled Copper.* Earl S. Bardwell. (56) Vol. 49, 1914.
The Layout of Concrete Products Plants.* E. S. Hanson. (110) Dec., 1914.

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Mechanical—(Continued).

Design of Surface Combustion Appliances.* Charles E. Lucke. (Paper read before the Am. Gas Inst.) (6) Serial beginning Jan.
Regulation of Jitney Busses in St. Louis. (60) June.
Boiler Failures and What the American Society of Mechanical Engineers is Doing to Prevent Them. E. R. Fish. (Paper read before the Engrs.' Club of St. Louis.) (1) July.
Uses of the Grinding Wheel.* B. F. Jacobs. (108) July.
English Motor Fire Engines.* (60) July.
Bearings of Electrical Machinery.* Andrew Gibson. (Paper read before the Assoc. of Min. Elec. Engrs.) (47) July 16.
Coking and By-Product Installation at Victoria Works, Ebbw Vale.* (22) July 16.
Waste-Heat Boilers in Steel Plants.* C. J. Bacon. (Paper read before the Am. Iron and Steel Inst.) (47) July 16.
Firebrick for Boiler Settings. William A. Heisel. (Abstract of paper read before the Ohio Soc. of Mech., Elec. and Steam Engrs.) (47) July 16.
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The Design and Construction of Aerial Ropeways.* (12) July 23.
Cracked and Seized Pistons on Diesel Engines.* Geo. E. Windeler. (Paper read before the Diesel Engine Users' Assoc.) (47) July 23; (64) Aug. 10.
The Automatic Loom.* (12) Serial beginning July 23.
Coal Handling at Panama.* (64) July 27.
Cranes for the Machine Shop and Foundry.* H. M. Lane. (20) July 29.
The Economic Forces Behind the Machine Tool Industry. Ludwig W. Schmidt. (72) July 29.
Log-Handling Equipment at Arrowrock Dam.* Charles H. Paul. (13) July 29.
Battery Vehicles as an Adjunct to Tramways. W. H. L. Watson. (Abstract of paper read before the Tramways and Light Railways Assoc.) (47) July 30.
Electric Vehicles for Collection of Town's Refuse.* J. Jackson. (Report to the Birmingham Corporation Refuse Disposal Dept.) (73) July 30.
Standardisation of B. A. Screws. (47) July 30.
The Theory of Lubrication.* (11) Serial beginning July 30.
Gas Operated Internal Combustion Engines. H. W. Edmund. (Paper read before the Illinois Gas Assoc.) (101) July 30.
Rerating a Pitometer.* Chester Gordon Gillespie.* (14) July 31.
Safety First, Its Application to the Design of Machine Fixtures.* Albert A. Dowd. (9) Aug.
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Comparative Test of Air Tools.* Dan Patch. (72) Aug. 5.
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The Case for the Electrification of Cane Sugar Mills and Refineries. Ernest P. Hollis. (26) Serial beginning Aug. 6.
Economics of the Jitney Bus Movement. F. W. Doolittle. (Abstract from *The Journal of Political Economy.*) (17) Aug. 7.

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The Electric Furnace in the Foundry. W. L. Morrison. (Paper read before the Am. Foundrymen's Assoc.) (47) Aug. 20.
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Digest of Jitney Ordinances. Clyde Lyndon King. (17) Aug. 21.
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Causes and Prevention of Clinker.* L. Rankin. (64) Aug. 31.
New Gas Plant at Budapest.* I. Bernauer. (83) Serial beginning Sept. 1.
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Versuche über die Grosse der wirksamen Kraft zwischen Treibriemen und Scheibe.* A. Friederich. (48) Serial beginning July 3.
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Fabrikanlage und Kühlhaus der Gross-Schlächterei und Wurstfabrik Bell A.-G. in Basel.* (107) Serial beginning Aug. 14.

Metallurgical.

The Treatment of Complex Ores by the Ammonia-Carbon Dioxide Process. S. E. Bretherton. (56) Vol. 49, 1914.
The Descriptive Technology of Gold and Silver Metallurgy. A. W. Allen. (56) Vol. 49, 1914.
The Dorr Hydrometallurgical Apparatus.* John Van N. Dorr. (56) Vol. 49, 1914.
The Slime-Concentrating Plant at Anaconda.* Frederick Laist and Albert E. Wiggin. (56) Vol. 49, 1914.
Development of the Round Table at Great Falls.* Arthur Crowfoot. (56) Vol. 49, 1914.
Unit Construction Costs from the New Smelter of the Arizona Copper Co., Ltd.* E. Horton Jones. (56) Vol. 49, 1914.
Chloridizing Leaching at Park City.* Theodore P. Holt. (56) Vol. 49, 1914.
Leaching Experiments on the Ajo Ores.* Stuart Croasdale. (56) Vol. 49, 1914.
Basic-Lined Converter Practice at the Old Dominion Plant.* L. O. Howard. (56) Vol. 49, 1914.
Lead-Matte Converting at Tooele. Oscar M. Kuchs. (56) Vol. 49, 1914.
Effects of the Bag House on the Metallurgy of Lead.* L. Douglass Anderson. (56) Vol. 49, 1914.
The Bag House in Lead Smelting.* H. H. Alexander. (56) Vol. 49, 1914.
Electrical Flume Precipitation at Garfield.* W. H. Howard. (56) Vol. 49, 1914.
The International Lead Refining Plant.* G. P. Hulst. (56) Vol. 49, 1914.
Lead Smelting at East Helena. Edgar L. Newhouse, Jr. (56) Vol. 49, 1914.
Electrostatic Separation at Midvale.* H. A. Wentworth. (56) Vol. 49, 1914.
Separation of Lead, Zinc, and Antimony Oxides. Richard D. Divine. (56) Vol. 49, 1914.

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Leaching Copper Products at the Steptoe Works. W. L. Austin. (56) Vol. 49, 1914.
Experimental Leaching at Anaconda. Frederick Laist and Harold W. Aldrich. (56) Vol. 49, 1914.
Precipitation of Copper from Solution at Anaconda.* Frederick Laist and F. F. Frick. (56) Vol. 49, 1914.
Melting of Cathode Copper in the Electric Furnace. Dorsey A. Lyon and Robert M. Keeney. (56) Vol. 49, 1914.
Economy and Efficiency in Reverberatory Smelting. C. D. Demond. (56) Vol. 49, 1914.
The Treatment of Copper Ore by Leaching Methods. W. L. Austin. (56) Vol. 49, 1914.
Curves for the Sensible-Heat Capacity of Furnace Gases.* C. R. Kuzell and G. H. Wigton. (56) Vol. 49, 1914.
A Comparison of the Huntington-Heberlein and Dwight-Lloyd Processes. W. W. Norton. (56) Vol. 49, 1914.
Nodulizing Blast-Furnace Flue Dust. Lawrence Addicks. (56) Vol. 49, 1914.
Smelting Lead Ores in the Blast Furnace. Irving A. Palmer. (56) Vol. 49, 1914.
Losses of Zinc in Mining, Milling, and Smelting. Dorsey A. Lyon and Samuel S. Arentz. (56) Vol. 49, 1914.
Furnace Curves.* R. J. Weitlaner. (105) July.
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Many High Speed Tool Steels.* Fred C. A. H. Lantsberry. (Paper read before the West of Scotland Iron and Steel Inst.) (20) July 29; (47) Aug. 20.
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Recent Blast Furnace Advancement. A. E. Maccoun. (Paper read before the Am. Iron and Steel Inst.) (62) Aug. 1.
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Note sur les Constructions Economiques ou Démontables.* G. Espitallier. (92) May.
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Mining.

The Ajo Copper-Mining District.* Ira B. Joralemon. (56) Vol. 49, 1914.
Mining Methods at the Copper Queen Mines.* Joseph P. Hodgson. (56) Vol. 49, 1914.
The Drumlummon Mine, Marysville, Mont.* Charles W. Goodale. (56) Vol. 49, 1914.
Copper Ores of the New London Mine.* B. S. Butler and H. D. McCaskey. (56) Vol. 49, 1914.

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Tests of Rock Drills at North Star Mine, California.* Robert H. Bedford and William Hague. (56) Vol. 49, 1914.
Methods and Economies in Mining.* Carl A. Allen. (56) Vol. 49, 1914.
Dip Chart (Used in Making Vertical Sections of Ore Deposits in Mines).* Howland Bancroft. (56) Vol. 49, 1914.
"Playa" Panning on the Conca River.* William F. Ward. (56) Vol. 49, 1914.
Mining Claims Within the National Forests. E. D. Gardner. (56) Vol. 49, 1914.
Rope Idlers in the Raven Shaft.* George A. Paskard. (56) Vol. 49, 1914.
The Design, Construction and Cost of Two Mine Bulkheads.* Sidney L. Wise and Walter Strache. (56) Vol. 49, 1914.
Ancient Auriferous Gravel Channels of Sierra County, California.* Mark N. Alling. (56) Vol. 49, 1914.
The Winding-Drums of Practice and of Theory; with Notes on Factors of Safety and Economy of Winding-Ropes.* H. W. G. Halbaum. (Paper read before the North of England Inst. of Min. and Mech. Engrs.) (106) Vol. 49, Pt. 3.
Notes on Sampling. T. W. D. Gregory. (Paper read before the North Staffordshire Inst. of Min. and Mech. Engrs.) (106) Vol. 49, Pt. 3.
Modern Developments in Hydraulic Stowing with Suggestions for Its Application in the Staffordshire and District Coal Field, and the Recovery of Abandoned Coal. J. Drummond Paton. (Paper read before the South Staffordshire and Warwickshire Inst. of Min. Engrs.) (106) Vol. 49, Pt.3.
An Auxiliary Aid Outfit for Attachment to Self-Contained Rescue-Apparatus.* Michael McCormick. (Paper read before the Min. Inst. of Scotland.) (106) Vol. 49, Pt. 3.
Rescue-Station Organization; the Resident Brigade System *versus* the System of Non-Resident Brigades. Henry Briggs. (Paper read before the Min. Inst. of Scotland.) (106) Vol. 49, Pt. 3.
The Drift-Deposits of Prestwich, Manchester and Neighbourhood.* J. E. Wynfield Rhodes. (Paper read before the Manchester Geol. and Min. Soc.) (106) Vol. 49, Pt. 3.
Mining in Burma.* C. W. Chater. (Paper read before the North of England Inst. of Min. and Mech. Engrs.) (106) Vol. 49, Pt. 3.
Steel Shaft Timbering at Los Ocotes Mine.* R. H. Cromwell. (6) Jan.
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The Potosi Tin-Mining District, Bolivia.* Francis Church Lincoln. (103) July 24.
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Effectual Sealing of Water Dams.* Frank H. Waterhouse. (Paper read before the National Assoc. of Colliery Managers.) (45) Aug.
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Bagley Scraper for Gravel Mining in Alaska.* Lewis H. Eddy. (16) Aug. 14.
The Wisconsin Zinc District.* H. C. George. (16) Serial beginning Aug. 21.
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Industrial Research. L. A. Hawkins. (From the *General Electric Review.*) (73) Aug. 6.
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Reinforced Concrete Pavements and Roadways. B. S. Pease. (110) Feb. 1914.
Some Essential Requirements in Concrete Pavements. R. C. Stubbs. (110) Feb. 1914.
Construction and Maintenance of New York State Highways. Arthur H. Blanchard. (6) Jan.
Proposed Revised Standard Specifications for Two-Course Concrete Street Pavements.* (Am. Concrete Inst.) (110) Jan.
Proposed Standard Specifications for One-Course Concrete Alley Pavements. (Am. Concrete Inst.) (110) Jan., Mar.
Proposed Revised Standard Specifications for One-Course Concrete Highway.* (Am. Concrete Inst.) (110) Jan.
Proposed Revised Standard Specifications for One-Course Concrete Standard Pavements.* (Am. Concrete Inst.) (110) Jan.
Review of Present Practice in Concrete Road Construction. Percy H. Wilson. (110) Mar.
Description of the Oxford Pike Service Test Concrete Roadway, Philadelphia.* William H. Connell. (110) Mar.
Concrete Roads and Frost Action. Andrew M. Lovis. (110) Mar.
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Some Maintenance Costs. Paul Macy. (36) May.
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Improvement of Sheridan Road in Highland Park, Ill.* Stanley E. Bates. (60) June.
Colloids in Relation to Manipulation of Structural Materials. Clifford Richardson. (From the *Harvard Technology Monthly.*) (60) June.
Financing and Building a County Highway.* (60) June.
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Progress of State Management of Public Roads.* J. E. Pennybacker. (Abstract of paper from Yearbook, Dept. of Agriculture.) (86) Aug. 18.
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The Counterbalancing of Locomotives.* S. G. Thomson. (Paper read before the Am. Ry. Master Mechanics' Assoc.) (18) July 31.
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Conditions Affecting the Success of Main Line Electrification.* W. S. Murray. (42) Aug.
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PAPERS AND DISCUSSIONS

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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THE HYDRAULIC JUMP, IN OPEN-CHANNEL FLOW AT HIGH VELOCITY

By Karl R. Kennison, Assoc. M. Am. Soc. C. E.

To be Presented November 3d, 1915.

Synopsis.

This paper presents an analysis of the hydraulics of the turbulent discharge below a spillway dam, where the so-called "jump" frequently occurs. A knowledge of the hydraulic principles involved should enable destructive high velocities and turbulence to be avoided, or intelligently provided for, in the design of flumes, dam foundations, etc.

The arrangement of the paper is as follows:

1.—Introductory.

2.—Conclusions.

The interesting feature is that in every open channel, except at controlling sections where the discharge is a maximum, there is, in addition to the existing water level, another level at which the same quantity of water might be flowing, under the same head. These two "alternative stages" should be recognized in the design of all structures for controlling the flow of water. The hydraulic jump is merely the turbulent passing between these two stages.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

1.—Introductory.

When water is discharged into a flume through a contracted gateway and under a considerable head, it sometimes continues to move in a thin sheet at a high velocity along the bottom of the flume for several hundred feet. Then it suddenly becomes turbulent and forms what is called a "hydraulic jump", the surface level down stream from this point being much higher than that of the approaching high-velocity discharge; or, when water flows over an ogee dam and out on a smooth apron it sometimes continues in a thin sheet, having a surface level far below the normal level of the river a little farther down stream, until it suddenly changes into a tumbling mass, rising to the normal river level by this "back roll" or "hydraulic jump". An excellent illustration of this is seen in Fig. 1, taken from a photograph, kindly furnished by the United States Reclamation Service, of certain flood conditions over the Granite Reef Dam.

This phenomenon sometimes becomes of great practical importance, and has been investigated mathematically by the writer at the request of John R. Freeman, M. Am. Soc. C. E., because of its interest in connection with the design of two important dams under widely different conditions in different parts of the country. In one of these cases it was desirable that the back roll or hydraulic jump should not be pushed far down stream from the foot of the ogee, off from a concrete apron which protected a clay river bed from scour; in the other case it was desirable that the back roll or hydraulic jump, with its violent surges, should not be pushed down stream so far as to interfere with the draft-tube exits of the power-house.

The problem does not appear to have received proper attention in the textbooks. No exhaustive mathematical analysis is attempted here, but merely an explanation of the peculiar conditions of flow which make the jump possible. The conclusions are as follows:

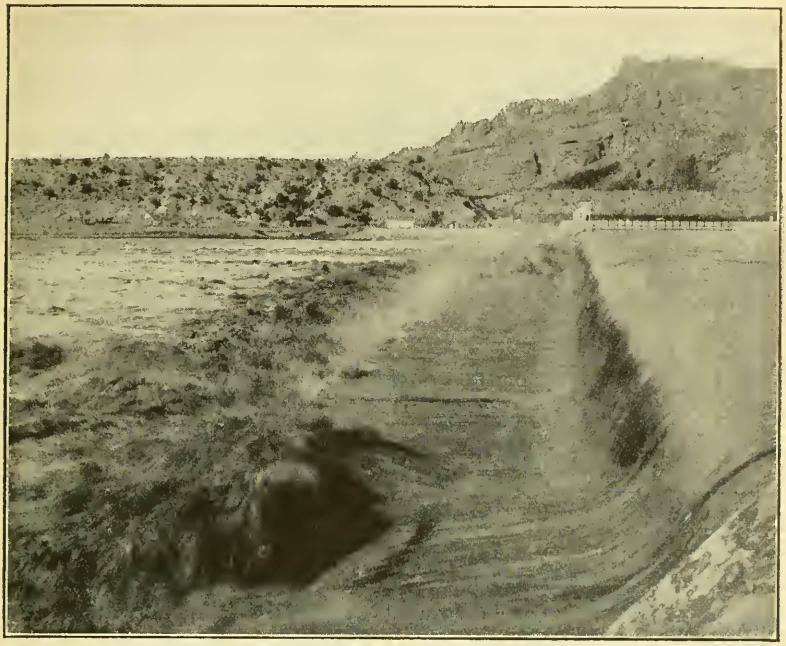

FIG. 1.—THE GRANITE REEF DAM IN ACTION.

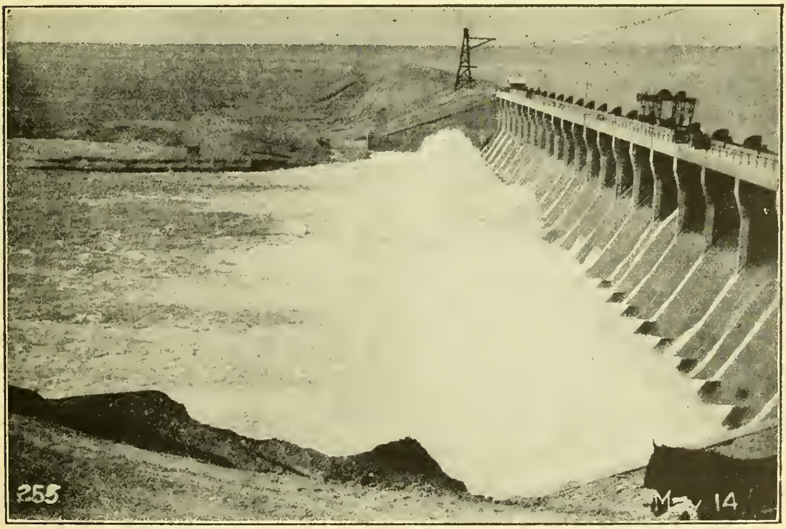

FIG. 2.—THE BASSANO DAM IN ACTION.

2.—Conclusions.

In the case of water flowing in an open channel on a steep gradient there are certain controlling sections which throttle the flow and determine the quantity of the discharge, that is, certain points where, for the given head and channel depth, the discharge is a maximum. If the contraction which causes this throttling of the flow is sufficiently gradual—for example, a submerged dam with smooth gradual approach and get-away—it can be shown that the depth of water at this point is theoretically two-thirds of the total head measured from the channel bottom or dam crest up to the hydraulic gradient, and the discharge per foot of length, therefore, should be $3.09\ H^{\frac{3}{2}}$.

At other points than at the controlling sections, however, the depth of water is not necessarily determined by the quantity discharged and the available head, but also by the channel conditions; because, for a given quantity of water flowing and a given head or elevation of hydraulic gradient, there are two possible surface-water levels which we will call alternative stages. The upper stage is the normal level in an ordinary stream, and for very low velocities is practically coincident with the hydraulic gradient. The lower stage is that ordinarily taken by water discharged at high velocity from an orifice or below a spillway dam. This is the more unstable of the two levels, due to the friction of high velocity on the channel bed. In other words, it can be shown that in any open channel, except at controlling sections such as just referred to, there is, in addition to the existing water level, another level at which the same quantity of water might be flowing with equal steadiness and under the same head or elevation of hydraulic gradient.

As these two definite alternative stages are the only possible ones under the existing head for smooth undisturbed flow, the stream must stand at one of these two levels, that is, water flowing in a smooth channel of uniform section must continue to flow at its existing stage, whichever one that happens to be, until, either due to a change in the channel bed or after sufficient loss of head in friction, it encounters a controlling section where the two alternative stages merge into one, and the depth is two-thirds of the total head. Below this controlling section the two possible stages again separate. At such a point as this the water level may change, without disturbance or interruption of the steady flow, from upper to lower stage, or from lower to upper

stage, or may continue at the same stage. For example, the water behind a spillway dam is approaching at the upper stage and just below the dam it flows away at the lower stage, the change occurring smoothly over the dam where the two stages were merged into one. On the other hand, if the dam is submerged by back-water almost as high as the up-stream pool, the surface may simply dip down locally at the dam where the depth is two-thirds of the head. In this case the upper alternative stage is maintained throughout, below as well as above the dam. It can also be shown that under certain circumstances the flow over a dam may theoretically be reversed, the dam facing down stream, and the water apparently running up hill—simply a case of passing from the lower to the upper stage smoothly, over a gradual controlling section where the depth is equal to two-thirds of the head.

In such phenomena the presence of the controlling section tends to eliminate any disturbance in passing from one level to the other, so that the existence of the two alternative stages is not noticed; but water flowing at the lower high-velocity stage and suddenly encountering obstructions which tend to destroy its velocity may rise suddenly, and with considerable disturbance and eddying, to the more stable upper low-velocity stage, and this phenomenon is the so-called "hydraulic jump", occasionally observed in open channels, and a common occurrence below spillway dams. It is merely the passing between the two alternative stages.

In this phenomenon the only energy loss is that due to the accompanying disturbance and eddying, the jump proper merely transferring kinetic into potential energy. Ordinarily, however, this hydraulic jump occurring below a spillway dam is accompanied by such violent disturbance and eddying that the total surplus energy in the water may be destroyed in this way. The jump proper, or the passing from lower to upper stage, does not involve energy losses, except incidentally, and it is doubtful if the ordinary formula for loss by "sudden expansion" applies in this case.

3.—Mathematical Basis for Conclusions.

(*a*) *Relation Between Depth, Head, and Discharge.*—The mathematical deductions by which the foregoing conclusions were reached

are presented with the assistance of diagrams and as briefly as possible so as to avoid confusing mathematical details. The careful reader will notice that in some parts of the discussion the variation of velocity at different points in the cross-section of a channel and the variation of the slope of the hydraulic gradient under different conditions of flow, have been neglected. This has been done for the sake of simplicity, and does not affect the general conclusions as to the existence of the two alternative stages or the proposition that the hydraulic jump is not an absorber of energy by "sudden expansion". Referring to Fig. 3, the gradual slope of the hydraulic gradient represents channel friction losses.

H = the total head above the channel bed or distance up to the average hydraulic gradient for all the water in the stream;
d = depth of water;
V = average velocity;
Q = quantity of water discharged per unit width of channel.

This assumption, that we are dealing with a strip of unit width, is made for the sake of simplicity, and the conclusions will not be directly applicable to the full width of the stream unless the form of its cross-section approaches a rectangle.

From the relations, $d + \frac{V^2}{2\,g} = H$, and $V = \frac{Q}{d}$, we obtain the following general expression or equation giving the relation between depth, d, head, H, and discharge, Q:

$$d^3 - H\,d^2 + \frac{Q^2}{2\,g} = 0.$$

Note that this is a cubic equation in d, that is, for a constant head and discharge, there are three roots or three values of the depth, d. One of these comes out negative, leaving two real values corresponding to the two alternative stages, as shown in the following example.

Suppose the head, $H = 10$ ft., and the discharge, $Q = 50$ cu. ft. per sec. The general equation then becomes $d^3 - 10d^2 + 38.8 = 0$. Solving,* $d = 9.58$, 2.24, or -1.82. The last root, being negative, is

* The writer has never seen a formula arranged to give directly the three roots of any cubic equation. Hence the following is presented in the hope that others may find it useful:

If $$x^3 + Ax^2 + Bx + C = 0$$

$$x = \frac{2}{3}\left(+\sqrt{A^2 - 3\,B}\right) \cos. \frac{1}{3} \cos.^{-1} \frac{9\,A\,B - 2\,A^3 - 27\,C}{2\,(+\sqrt{A^2 - 3\,B})^3} - \frac{A}{3}.$$

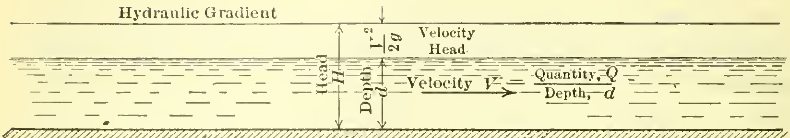

FIG. 3

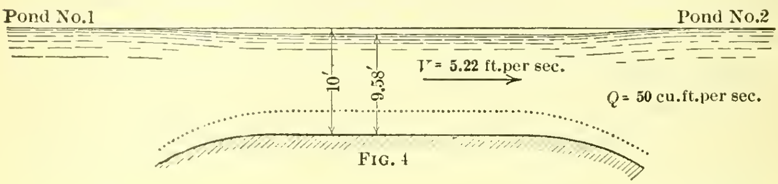

FIG. 4

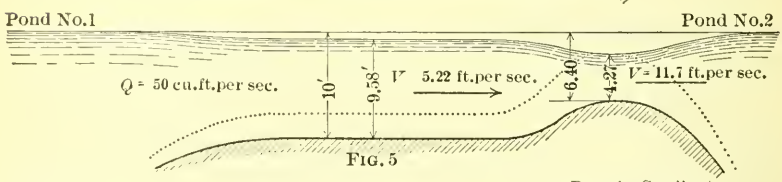

FIG. 5

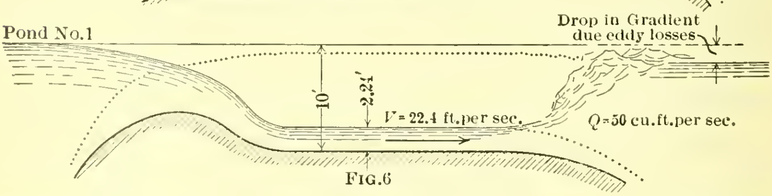

FIG. 6

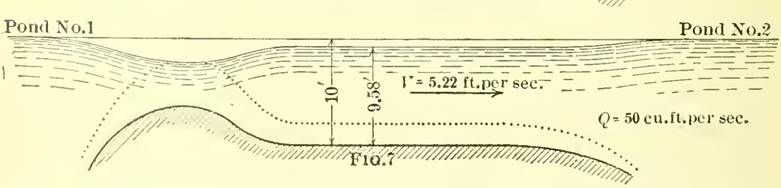

FIG. 7

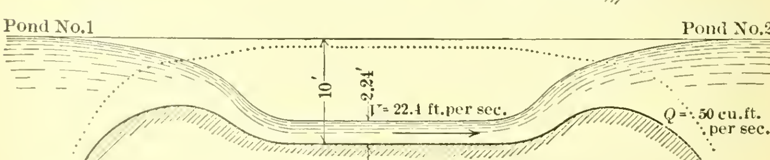

FIG. 8

unreal, and the two alternative stages correspond to depths of 9.58 and 2.24 ft., respectively. This condition is shown in Figs. 4 to 8, in which 50 cu. ft. per sec. are discharged at either of the two stages and under the same head of 10 ft. In these diagrams, for convenience, the channel is represented as a short rectangular flume between two large ponds. Pond No. 1 at the left empties into Pond No. 2 at the right, the entrance and exit of the flume converging and diverging gradually and smoothly so as to prevent eddy losses of energy. In each illustration the fine upper line is the hydraulic gradient, the water surface is shaded, and its alternative stage is shown dotted.

(*b*) *Maximum Discharge at Controlling Section.*—The assumptions as to the head and discharge, however, were purely arbitrary, and channel conditions would have to be such as to enable this discharge of 50 cu. ft. per sec. to be maintained under this head. For example, either the gradient between Ponds 1 and 2 would have to be so slight, or the channel so rough, as to limit the velocity to 5.22 ft. per sec., $Q = 50$ cu. ft. per sec. (Fig. 4), or else the flow would have to be throttled down to 50 cu. ft. per sec. by the controlling sections shown in Figs. 5 to 8. Otherwise, if these dams were removed, the flume itself would become the controlling section, and the discharge would be increased to the maximum carrying capacity of the flume.

To find what this maximum discharge is at any controlling section, rearrange the general equation, $d^3 - H\,d^2 + \frac{Q^2}{2\,g} = 0$, so as to obtain $Q = \sqrt{2\,g\,(H\,d^2 - d^3)}$. Differentiating this with respect to d, we find that Q is a maximum when $d = \frac{2}{3}\,H$, and substituting, we find $Q = 3.09\,H^{\frac{3}{2}}$. Also, under these conditions, the two positive roots of the foregoing general equation are equal, that is, the two alternative stages merge into one, as shown by the shaded and dotted lines in the diagrams. Hence, it follows that at any such gradual controlling section in an open rectangular channel, so long as the drop in gradient is sufficient to limit the discharge only by the available head above the channel bed, then the depth, $d =$ two-thirds of this total H, measured above the channel bed, and the discharge Q equals $3.09\,H^{\frac{3}{2}}$.

This is hardly to be taken as a practical formula for actual weir discharge, as the general equation for open-channel flow, on which it

is based, assumes that the pressure gradient coincides with the water surface, or that the water in every part of the section is under a pressure head equal to the vertical height of water above it. As a matter of fact, on a sharp-crested weir, or on a dam crest, or similar controlling section, the under side of the over-falling discharge is at atmospheric pressure, or approximately so. This means an increase in the velocity head throughout the section, and hence the discharge, Q, will exceed $3.09\ H^{\frac{3}{2}}$ per foot of width. Note that the Francis formula for a sharp-crested weir is $Q = 3.33\ H^{\frac{3}{2}}$ per foot of width.

The formula, $Q = 3.09\ H^{\frac{3}{2}}$, would hold in the case of a by-pass flume with smooth "bell-mouth" entrance and free get-away at the down-stream end, or for a smooth, gradually-formed, submerged dam, where the direction of flow does not change so suddenly as to alter the conditions of internal pressure. In fact, for convenience in illustrating, the dams in Figs. 5 to 8 have been drawn with a contracted horizontal scale. They would really need to have a much broader base.

4.—Examples of the Two Alternative Stages.

Fig. 9, in which one of these controlling sections is drawn more nearly to scale, shows very clearly the existence of the two alternative stages. The water level was drawn flowing up hill from lower to

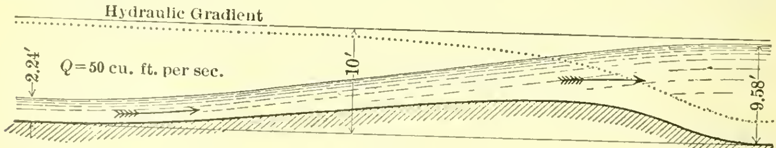

Fig. 9

upper stage, and then a suitable controlling section was computed which would allow this condition of flow. This illustrates the recovery of velocity head smoothly without any loss of energy. If the discharge was reduced to a little below 50 cu. ft. per sec., the two alternative stages would not meet, and the water would "jump" the intervening space to reach the more stable upper level. This would be easily accomplished, for it can be shown that although the head, or energy content, remains constant, yet at or near the critical stage of maximum discharge, where the depth is equal to two-thirds of the head, the water surface is comparatively uncertain, and is easily subject to considerable fluctuation for very slight inequalities in effective area of

section. This tendency may be noted in a long flume under maximum discharge, such as a flood by-pass flume around a coffer-dam. It has this inherent tendency to surface fluctuations, which should not be aggravated by bends in the flume where it is possible to make the latter perfectly straight.

The existence of the two stages is also brought out in Figs. 6 and 7. Referring to Fig. 6: In case Pond No. 2, into which the flow empties, is low enough—so that, after the friction losses in the flume and all the eddy losses at the flume exit, there is still enough potential energy left to reach the pond level—then conditions will remain as in Fig. 6, with either a definite "hydraulic jump" as shown, or a series of more or less irregular waves, finally reaching the level of Pond No. 2. But if Pond No. 2 is higher than this, that is, so high that it cannot be reached by the total potential energy of the flume discharge, after the necessary friction losses of the high velocity in the flume and eddy losses caused by the jump have been deducted, then the pond will back up into the flume somewhat as shown in Fig. 7.

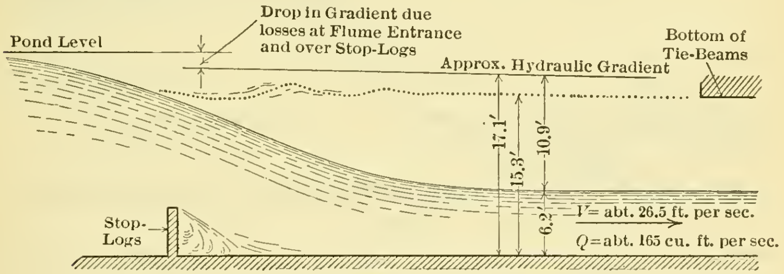

FIG. 10

This is apparently what happened in the case of a flume of the Massachusetts Metropolitan Water-Works, observed and reported by F. P. Stearns, Past-President, Am. Soc. C. E. Fig. 10 is taken from a print showing the measured velocities and surface profile in this flume, 40 ft. wide, emptying a reservoir at the left. The water is apparently flowing at the lower alternative stage, and the corresponding upper alternative stage is shown dotted. Besides what is shown in this figure the print shows the flume extending to the right a total length of 671 ft., with a bend in the middle which caused irregular waves in the down-stream half of the flume. Mr. Stearns states that during a later flood than the one shown here—possibly a greater flood, but only

slightly greater—he visited the flume and found the water at just about the level marked "bottom of tie-beams". Note the coincidence between the upper alternative stage and this point at which Mr. Stearns observed the water surface. Apparently, the discharge was not materially different in the two floods, but in the one case the particular channel conditions maintained the lower alternative stage and in the other the upper alternative stage.

5.—Examples of the Hydraulic Jump.

The gradual throttling of the flow represented in Figs. 5 to 8 permits the discharge to follow along either the lower or upper alternative stage, as shown particularly by comparing Figs. 6 and 7, but, in the case of the ordinary dam crest, the contraction is so sudden that the conditions of internal pressure are affected. The tendency is all

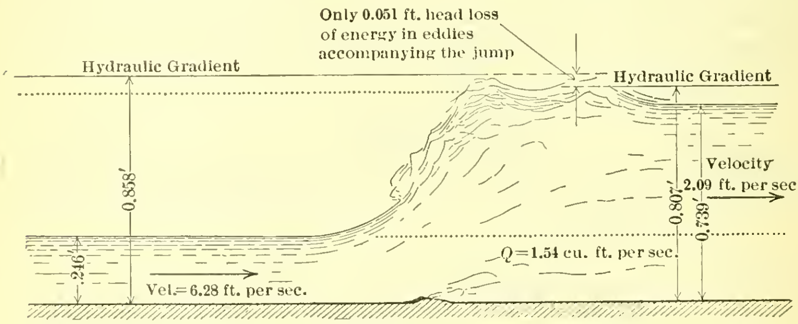

Fig. 11

toward a decrease in the pressure throughout the overfalling sheet of water, and a consequent increase in the velocity. Hence a weir or dam furnishes a velocity-creating condition, and tends to force the water level to drop to the lower or high-velocity stage, where it must continue until it encounters a velocity-destroying and pressure-creating condition, when it will rise again to the more stable upper stage of low velocity.

This rise is shown in Fig. 11, which represents one of four observations of the hydraulic jump by Bidone, quoted by Merriman as follows: The depth before the jump equals 0.246 ft.; the velocity before the jump equals 6.28 ft. per sec.; the depth after the jump equals 0.739 ft.; from these data the other quantities shown in Fig. 11 are easily computed closely enough for our purposes, assuming that the depths

before and after the jump were measured from the same floor line, parallel to the hydraulic gradient. An actual case similar to this, and one which brings out the importance of recognizing the two alternative stages, is illustrated in a recent issue of a technical journal.* A concrete chute, 9 ft. wide and 5 ft. high, at the end of the Truckee Main Canal was designed to discharge the canal flow at high velocity into the Lahontan Reservoir; but, when the water was turned in, it was found that with small discharges a vertical curve at the bottom of the chute was sufficient obstruction to cause the water to jump to the upper low-velocity stage, which was so high that it began to overtop the concrete sides and had to be retained by sand bags before the discharge increased sufficiently to maintain the lower stage throughout the length of the chute.

6.—Destructive High Velocity Below Spillway Dams.

The jump shown in Fig. 11 illustrates what happens down stream from a spillway dam, except that in the case of a high spillway dam the elevation of the gradient up stream from the dam is fixed at the pond level and down stream from the dam it is fixed by the natural stream. The fixed difference between the two, or the net fall at the dam, is so much greater than the drop shown in Fig. 11 that the water cannot easily be made to jump to the upper stage until enough of the superfluous energy has been destroyed in friction, etc., so that the remainder of the drop can be absorbed by the eddies and disturbance accompanying the jump. Take, for example, the case of the very high dam shown in Fig. 12. As in previous figures, the dotted line represents the stage alternative to the water surface. The friction under the excessive velocity causes the hydraulic gradient to slope down steeply, and the two alternative stages to approach each other until they are near enough for the jump to occur between them, with its accompanying eddies destroying the remainder of the energy, as already explained. The horizontal scale is contracted, due to the limits of the size of the drawing.

In Fig. 12 the profiles of the hydraulic gradient and alternative stages are more or less arbitrarily drawn, but they illustrate the great vertical distance between the two theoretic alternative stages, and the consequent tendency of the destructive high-velocity flow to continue

* *Engineering Record,* April 10th, 1915.

some little distance down stream from the dam. This is further illustrated in Fig. 1, showing the Granite Reef Dam of the Salt River project of the U. S. Reclamation Service.

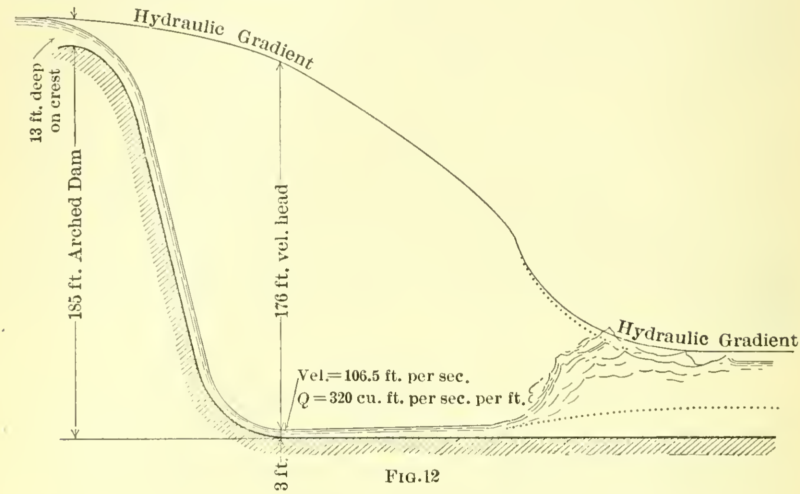

FIG. 12

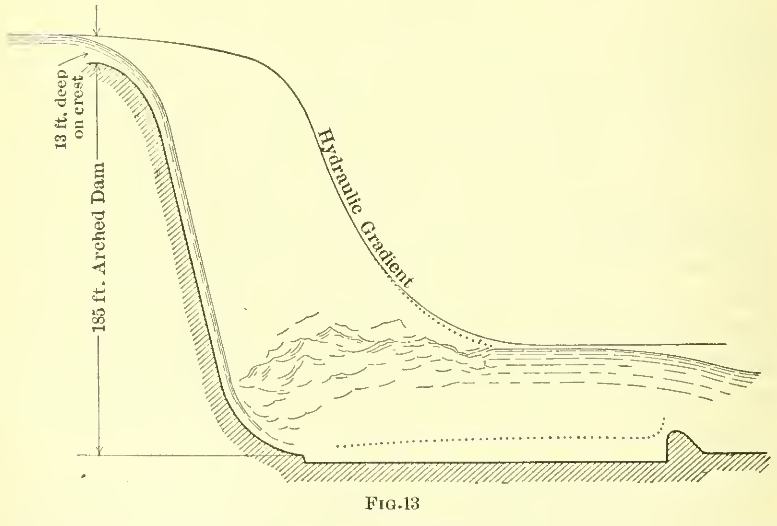

FIG. 13

Experience with actual spillways, as well as experiments on models, have shown that this tendency is counteracted, and the jump apparently brought back to the toe of the dam, if the pool into which the water plunges is deep enough. For example, as illustrated in Fig. 13, the

water-cushion is so deep, and the destruction of energy when the water strikes it is so great, that the total head is absorbed in this way, with no available energy left to maintain the high-velocity flow of Fig. 12.

The question arises: What is the necessary depth of pool to absorb the energy of a spillway discharge; or, in general, what are the conditions which will insure the occurrence of the hydraulic jump? The writer has noted that Merriman's original formula for the hydraulic jump, which is of doubtful accuracy and has been discarded by Merriman himself in his later works, has sometimes been applied to conditions below a spillway dam.* This formula is $d_2 = 2\sqrt{d_1 \frac{V_1^2}{2g}}$, in which d_1 and V_1 are the depth and velocity before the jump, and d_2 is the depth after the jump. Applying the formula to the actual jump shown in Fig. 11, we compute $d_2 = 0.777$ ft., which agrees fairly well with the observed depth after the jump, 0.739 ft. However, there is an apparent inaccuracy in the derivation of the formula, in that the head necessary to lift the water from d_1 to d_2 is taken to be only half the height of the jump, instead of the full height. Also, the formula is based on the assumption that there is a loss of head in "sudden expansion" equal to $\frac{(V_1 - V_2)^2}{2g}$, which in this case would amount to 0.273 ft., although the actual loss (see Fig. 11) was only 0.051 ft. Hence the reason for the apparent agreement between the computed (0.777) and the observed (0.739) depths is that in this particular case it took about 0.25 ft. more head to lift the water to the upper level than called for by the formula, and about 0.22 ft. less head was destroyed in eddies or absorbed. Possibly these two discrepancies might not come so near to balancing each other under entirely different conditions of flow. Hence, before using this formula to find a proper pool depth below a spillway dam,* one should consider whether or not it will apply under those conditions.

Certain unfinished and unpublished experiments on a model with linear dimensions one-forty-eighth the size shown in Fig. 13, indicate that, for these particular conditions also, at least in the model, the required amount of energy is destroyed and Merriman's original

* This use of the formula is not mentioned by Merriman, but has been made by other engineers. The latest edition of Merriman's "Hydraulics" treats the problem of the hydraulic jump differently.

formula gives approximately correct results, any pool depths greater than the computed 46 ft. causing the jump to occur at the toe of the dam and apparently preventing the water from shooting out straight at high velocity. Nevertheless, a few experiments not covering all conditions of head and discharge, do not prove the general applicability of a formula which is not based on correct theory.

The fact remains that the destructive energy due to the drop over a spillway dam is a definite, fixed quantity, regardless of the presence or absence of any hydraulic jump. The only destruction of this energy (conversion into heat) in the hydraulic jump is that due to the accompanying eddies and disturbances, and is measured by the drop in hydraulic gradient. If this drop is large, as over a high spillway dam, the disturbance is equally large, and unavoidable. The Bassano Dam of the Canadian Pacific Railroad Irrigation Project is equipped, after the recommendation of Mr. John R. Freeman, with two staggered rows of "baffle-piers", shaped like snow-plows, pointed up stream, and designed to split up the high-velocity sheet of water before it can strike the bed of the stream, and throw one jet against another so that the energy will be absorbed as much as possible by eddies within the body of the down-stream pool, and not by tearing the foundations. These baffle-piers, backed up by a water-cushion, give assurance that the jump will start at the toe of the dam. They themselves are not designed to destroy the energy by impact, but merely to start the necessary eddies, which act in the water-cushion below the baffle-piers and complete the hydraulic jump.

The object of this paper has been to call attention to the existence of the two "alternative stages" in open-channel flow, and to their practical importance, in many cases, when the flow is at high velocity; it is also intended to identify the "hydraulic jump" as the passage from the lower to the upper stage. Below a spillway dam, it often happens that these two stages are so far apart that the jump cannot occur between them, until a considerable distance down stream and after enough energy has been destroyed in friction to bring the two stages near together, as explained in the paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

A STUDY OF THE DEPTH OF ANNUAL EVAPORATION FROM LAKE CONCHOS, MEXICO

By Edwin Duryea, Jr., M. Am. Soc. C. E.

and

H. L. Haehl, Assoc. M. Am. Soc. C. E.

To be Presented November 17th, 1915.

Synopsis.

This paper treats of the determination of the yearly evaporation depth from a large reservoir, in an instance where it was of unusual importance to ascertain a safe and reasonably accurate value, where there were no existing evaporation data for places nearer than 300 miles, and no local data except mean temperatures and elevation above sea level. It treats also of the checking of the depth thus estimated by local evaporation observations started for that purpose.

Lake Conchos is a large artificial reservoir, about 300 miles south of El Paso, Tex., with an area (at spillway level) of 67.7 sq. miles, a storage capacity of 2 550 000 acre-ft., a maximum depth of 223 ft., and an average depth of 61 ft. The lake is about 4 300 ft. above sea level, and the mean yearly temperature of the region is about 67° Fahr.

The waters of the lake are to furnish electric power to the Parral mining district, about 50 miles distant; and the project's dams,

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings,* and, when finally closed, the papers, with discussion in full, will be published in *Transactions.*

power-house, transmission line and Parral Sub-station already (May, 1915) are so nearly completed that power will be delivered at Parral during the present summer.

Due to the great lake area and the probable great depth of yearly evaporation, the evaporation losses from this reservoir are of unusual importance as an element affecting the net water supply available for power production; and since widely differing opinions had been given as to the yearly evaporation depth, it was necessary to ascertain for it a safe and reasonably accurate value.

The yearly evaporation depth from the lake was estimated by three methods, *A*, *B*, and *C* (*B* and *C* each having two variants).

Method *A* is based on data of pan evaporations, mean temperatures, and elevations above sea level—all as observed at six stations in Texas and New Mexico, distant from 300 to 500 miles from Lake Conchos. It is also based on the known elevation of the lake, and the temperatures there, as observed for a period of about 2 years.

Method *B* consists of a modification and amendment of the yearly evaporation depth, estimated by Method *A*, by the aid of first 2 months' and later 5 months' observations of pan evaporations at Lake Conchos.

Method *C* consists of the estimation of the yearly evaporation depth from Lake Conchos by means of the excess of the inflow into the lake above the outflow—first for a period of 4 months and finally for a period of 7 months.

The yearly evaporation depths from the lake, estimated by the three methods, are as follows:

Method *A*				52.5 in.
Method *B*	(2 months) B_a =	52.9 in.;	B_b =	52.4 in.
	(5 ") " =	54.1 " ;	" =	53.4 "
Method *C*	(4 months) C_a = less than	55.4 in.;	C_b = less than	56.2 in.
	(7 ") " = " "	57.7 " ;	" = " "	59.3 "

At the end of the 2 months' observations, the value of 55 in. was adopted; and, as the final conclusion from the whole investigation, 55 in. was re-adopted, unchanged, as the safe yearly evaporation depth.

Besides the foregoing principal conclusions of this investigation, there are statements of the relative evaporation depths from pans

from 2 to 6 ft. square, data and conclusions as to the relative evaporation depths from land-pans and from floating-pans of the same size, and data and conclusions as to the effect of mean temperature and of elevation on the depth of evaporation.

Also (and this constitutes the most important feature of the paper), it is estimated by a combination of the results of Methods *B* and *C* that at Lake Conchos the evaporation depth from the lake surface certainly is less than 67.5% (and in all probability as little as 62%) of that from a pan 3 ft. square floating thereon. In 1910 Professor Frank H. Bigelow, of the United States Weather Bureau (in connection with his Salton Sea evaporation experiments), deduced a value of 62% for this coefficient; and, so far as known to the writers, the foregoing value at Lake Conchos is the only check of Professor Bigelow's value which ever has been made.

General.

Lake Conchos is an artificial reservoir in the State of Chihuahua, Mexico, about 20 miles west of the City of Santa Rosalia, 90 miles south of the City of Chihuahua, and 300 miles south of El Paso, Tex. The lake is on the Continental Plateau to the east of the Western Sierra Madre Range, is at an elevation of about 4 300 ft. above sea level, and is in a region having a mean temperature of about 67° Fahr. Its climate is very similar to that of El Paso, Tex., which is at an elevation of 3 700 ft., and has a mean temperature of about 63° Fahr.

The shape and extent of the lake are shown by the map, Fig. 1, and the general character of the shores and of the surrounding country is shown by the panoramic photograph, Fig. 2.

The lake or reservoir is formed by impounding the waters of the Rio Conchos (a tributary of the Rio Grande) by three dams—the La Boquilla Dam across the Rio Conchos, a masonry structure of 244 ft. maximum height and 840 ft. crest length; the Tigre Hill Dam, an auxiliary dam of 90 ft. maximum height and 2 780 ft. crest length; and the Babizas Spillway Dam, of 31 ft. maximum height and 3 190 ft. crest length. The La Boquilla Dam is a curved masonry structure of heavy gravity section (proportioned for "uplift"); the Tigre Hill is a masonry dam of inadequate gravity section, strengthened by an earth embankment against its down-stream face; and the Babizas

Dam is to consist of a heavy, gravity-section, masonry spillway, 2 360 ft. in length, flanked at its ends by earth embankments. The La Boquilla Dam is of cyclopean rubble laid in Portland-cement concrete. It is now (May, 1915) about 97% completed, and should be entirely completed within a month or two. The Tigre Hill is of hand-laid rubble in hydraulic-lime mortar, and is now nearly completed. The Babizas Dam is to be of hydraulic-lime concrete, and only its foundation excavation has been done as yet. The La Boquilla Dam now is sustaining a maximum depth of water of about 190 ft., and there is a maximum depth of about 15 ft. of water against the Tigre Hill Dam.

The plans for the three dams were made by William B. Fuller, M. Am. Soc. C. E. (formerly Chief Engineer), after general plans made by John R. Freeman, M. Am. Soc. C. E., Mr. Freeman's plans being restricted in some ways because of the necessity of making use of parts of the work previously designed and constructed by Messrs. S. Pearson and Son, the contractors during its early stages. The major part of the work as yet constructed was done by company labor, under the supervision of Mr. Fuller, who was the Chief Engineer of the Mexican Northern Power Company, Limited, from the fall of 1911 until August, 1913. The work is being completed by company labor, under the supervision of the present Chief Engineer, G. G. Underhill, Assoc. M. Am. Soc. C. E.

The purpose of the reservoir and of the project is to supply electric power to the Parral mining district, about 50 miles distant, from a power-house just below the La Boquilla Dam. This power-house, including its machinery, already is practically completed, and also the transmission line and the Parral sub-station; and contracts were made recently to deliver electric power at Parral during the summer of 1915.

Evaporation.

Toward the end of 1913, the writers found it necessary to investigate the evaporation losses from Lake Conchos, as one of the elements of a general investigation of the partly constructed power project. The lake is to be one of the largest artificial reservoirs in the world, its surface areas and storage capacities being as given in Table 1.

Its upper 78.8 ft. (the portion available for power production) has a storage capacity of 2 047 000 acre-ft. The waters stored below Eleva-

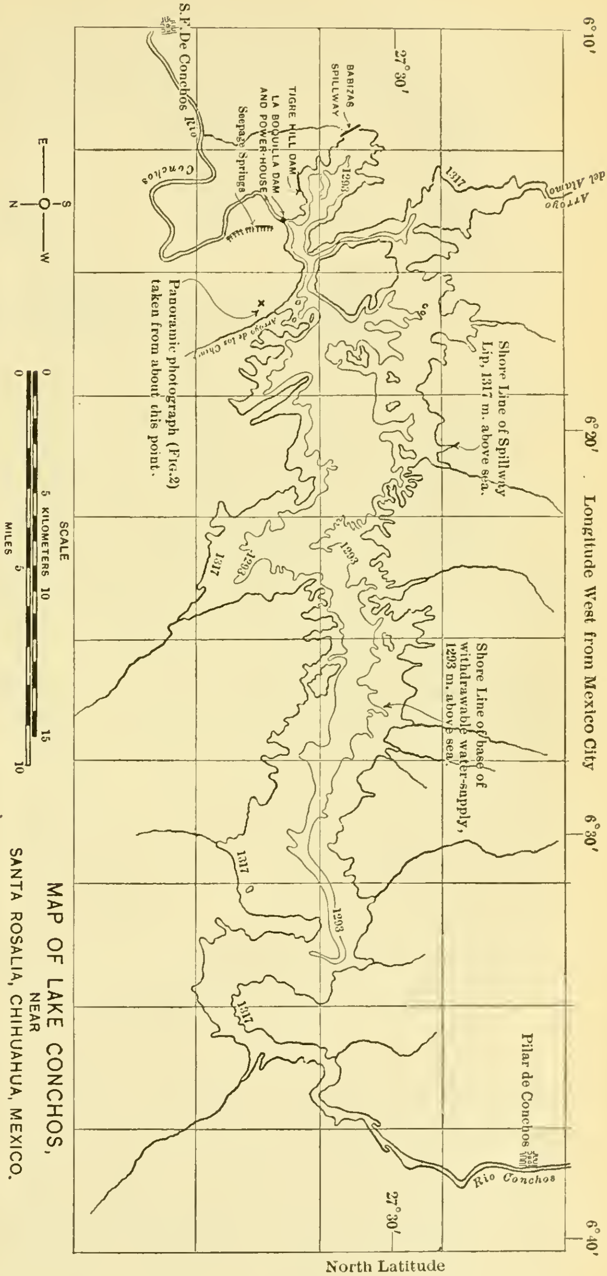


FIG. 1.

FIG. 2.—PANORAMIC VIEW LOOKING SOUTH ACROSS LAKE CONCHOS, FROM THE POINT MARKED x ON FIG. 1. FROM A PHOTOGRAPH TAKEN SEPTEMBER 15TH, 1914; WITH LAKE SURFACE AT ELEVATION 1303.5 M. ABOVE SEA LEVEL AND 44.3 FT. BELOW ELEVATION OF SPILLWAY LIP.

TABLE 1.—AREAS AND VOLUMES OF LAKE CONCHOS.

ELEVATION OF LAKE SURFACE ABOVE SEA LEVEL.		Area of lake surface. in square miles.	Storage capacity below lake surface. in acre-feet.
In meters.	In feet.		
* 1 317.0	4 319.8	67.7	2 550 000
† 1 312.5	4 305.0	55.3	....
‡ 1 293.0	4 241.0	22.0	503 000

* Elevation of lip of spillway.

† Probable average elevation of lake surface during long periods of full or ultimate power production (based on study of a 62-year period).

‡ Base of the stored waters withdrawable for power production.

tion 1 293 m., with a maximum depth of 144.4 ft., are useful only in supporting the waters above Elevation 1 293 m. at static heads of from 141.0 to 219.8 ft. above the tail-water elevation of 1 250 m.

Because of its unusually large surface area, the evaporation losses from the lake also will be of unusual magnitude and importance. In 1911 it was judged, without special investigation, by John R. Freeman and F. P. Stearns, Members, Am. Soc. C. E., that its average annual evaporation loss would be about 55 in. depth (Mr. Freeman) and 83 in. (Mr. Stearns)—the 55 and 83 in. being the supposed gross evaporations, unreduced by the rain falling directly on the reservoir surface. The 55 in. annual evaporation loss corresponds to an average loss from the lake surface at spillway level of 275 sec-ft.; and the 83 in. corresponds to an average loss of 415 sec-ft. These two values for the average evaporation loss vary widely from each other, and even the smaller is a greater proportion of the total stream flow than is usual in water-supply investigations. Hence, as one of the steps necessary to the reliable estimation of the safe net water supply available for power production, the estimation within reasonably close limits of the true evaporation loss from the lake was of much more importance than is usual in evaporation studies.

Evaporation data in Mexico were entirely lacking; there was a very close agreement between the values of the annual evaporation depth from Lake Conchos, as estimated by the writers by five different methods; and the average yearly evaporation depth, adopted after only 2 months' local evaporation tests, was identical with the depth adopted after 5 months' local tests; and for these reasons it is believed that this study of Lake Conchos evaporation possesses sufficient interest

and value to the Engineering Profession to justify the presentation of this paper.

The paper will consist of a preliminary and general description of the data and methods used in the investigation and of the results and conclusions derived therefrom, followed (as an Appendix) by the detailed data and operations leading up to those conclusions.

The general descriptions of the data and methods used, and of the resulting conclusions, are as follows:

Data.—The evaporation data on which the investigation is based are given in full in the Appendix. The unmodified data are given in Tables 17, 34, 36 (which are Plates XXXI, XXXV, and XXXVI), Figs. 3 and 14, and Table 37.

Figs. 4 to 13, inclusive, Fig. 15, Plates XXXII, XXXIII, and Table 21 (Plate XXXIV) deal with these data as modified and combined by the various methods of investigation used.

These data may be divided into two general groups: one includes all the obtainable data of evaporation depths and mean temperatures at stations in Texas and New Mexico; the other includes the evaporation observations initiated and carried out under the direction of the writers at La Boquilla, in conjunction with observations of mean temperature. These data comprise evaporation observations made in both land-pans and floating-pans in both groups, and include also the Piche evaporimeter observations made in Texas and New Mexico in 1887-88.

Texas and New Mexico Group.—All the data of measured evaporation depths in Texas and New Mexico were copied from such official or semi-official sources as the publications of the U. S. Weather Bureau, the U. S. Geological Survey, the University of New Mexico, *Engineering News*, Turneaure and Russell's "Public-Water-Supplies", etc., etc., the sources of all the data being given in Table 17 (Plate XXXI) in connection with their presentation. Such data of evaporation depths in Texas and New Mexico, measured in pans, as were made use of in this investigation are for the following stations and durations:

At Austin, Tex.—for the 12 months of 1911.

At El Paso, Tex.—for 34 months in 1889-90 to 1892-93.

At Albuquerque, N. Mex.—for 30 months in 1900-01 and 1903-04.

At Carlsbad, N. Mex.—for 13 months in 1899 and 1901.
At Elephant Butte, N. Mex.—for 6 months in 1909-10.
At Lake Avalon, N. Mex.—for 5 months in 1909-10.

Hence the measured evaporation data of Texas and New Mexico comprise records at six places, varying from 5 to 34 months at a place, and in all aggregating 100 months. It should be noted that all the six stations, like La Boquilla and Lake Conchos, are on the Great Plateau to the east of the Continental Divide (the Western Sierra Madre).

All the evaporation records in Table 17 (Plate XXXI) are given there for full years, such months as were not actually measured having been filled in by deduction. All such deduced monthly evaporation-depths are indicated in Table 17 (Plate XXXI), and their method of deduction and their probable errors are discussed in the Appendix.

At some of the above-named stations the evaporation measurements were made in land-pans, at others in floating-pans, and at others in both; and the pans used were of various sizes. The relative evaporation depths from pans of different sizes and from floating-pans in comparison with land-pans, are discussed fully in the Appendix; and corrections are there applied to bring the various evaporation depths to a uniform basis, so far as practicable, for all the stations, and thus permit of fair comparisons. It is believed that, as thus corrected, the various measured evaporation data in Texas and New Mexico possess sufficient accuracy to make them reliable for the object of this investigation.

The Piche evaporimeter measurements made use of in this investigation are those taken at each of eleven stations in Texas and New Mexico continuously throughout 12 months—after a somewhat disconnected study of this method had been made elsewhere for a longer period, to determine the relations between evaporation depth, temperature, vapor pressure, and barometric pressure. The evaporation depths at eleven stations, as deduced from the evaporimeter observations, also are presented in Table 17 (Plate XXXI). Subsequent comparative measurements of evaporation depths in pans, at scattered stations throughout the United States, are said to indicate that the Piche method gives somewhat greater evaporation depths than are

obtained from measurements in floating-pans—though the size of pan to which that statement applies is not known.

La Boquilla Group.—The evaporation measurements made at La Boquilla (Lake Conchos), and used in this investigation, comprise the measurements of evaporation depths made at the lake in two land-pans and two or one floating-pans from about the middle of January to early in June, 1914. The evaporation measurements for this whole period of about 5 months were available at the close of the evaporation study; but, to prevent delay in the progress of the water-supply investigation, it was necessary to adopt tentatively a yearly evaporation depth from Lake Conchos in March, 1914, at a time when only about 2 months' evaporation measurements had been made at La Boquilla. It is worthy of remark that the average yearly evaporation depth on the lake, as adopted tentatively from the Texas and New Mexico evaporation data in conjunction with the 2 months' measured evaporations at La Boquilla, was confirmed later, and almost exactly checked, by the longer evaporation measurements extending over about 5 months.

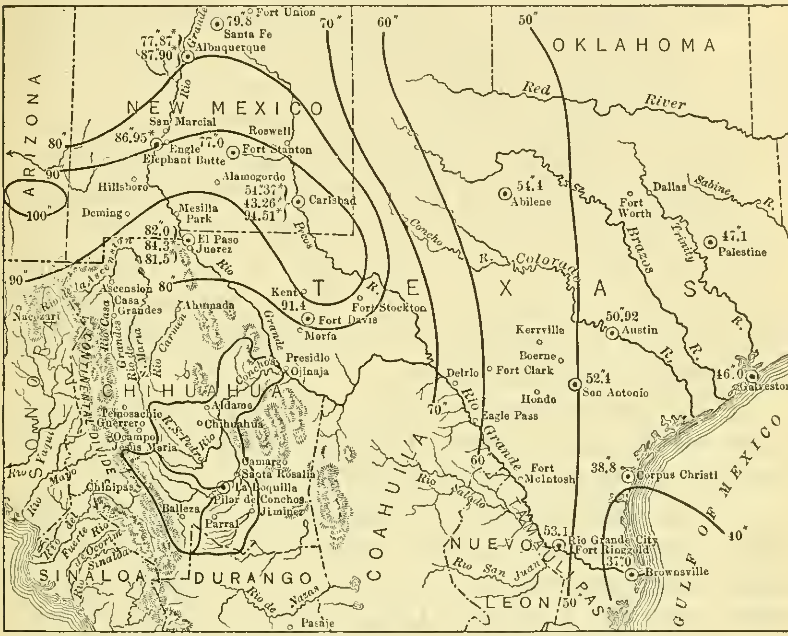
The evaporation measurements made at La Boquilla are given in Tables 34 and 36 (Plates XXXV and XXXVI).

There also were available for the investigation (see Table 37 and Fig. 14) the observed daily maximum temperatures at La Boquilla from April, 1912, to December, 1913, inclusive, the minimum temperatures from August, 1912, to August, 1913, and both maximum and minimum daily temperatures for January to June, 1914; and maximum and minimum daily temperatures at El Paso throughout 1912 and 1913.

As a rough general guide to evaporation depths in northern Mexico, there was available also a map showing probable lines of equal evaporation in Texas and New Mexico. This map, Fig. 3, indicates that the probable annual evaporation depth at Lake Conchos is between 70 and 80 in., presumably as measured in floating-pans.

Finally, in addition to the above-described La Boquilla, and Texas and New Mexico evaporation and temperature data, there were available the fluctuating surface elevations (and hence the net storage) of Lake Conchos, and the inflow and outflow of the lake, all as observed with reasonable closeness throughout the 7 months (practically rainless and with a nearly constant lake level) from October,

1913, to April, 1914, inclusive. These were used as data for an approximate estimate of the evaporation depth during those months, by deducting the net storage during the period from the excess of inflow



NOTES:- This Map shows all stations in Southern Texas and New Mexico at which evaporation records were available, with the inches of annual depth of evaporation measured or calculated for each.

The records are of two types: depths actually measured in evaporation pans, shown thus (50.92); and calculated evaporations from meteorological observations and Piche Evaporimeter records (made by the U.S. Signal Service in 1887-88 and published in the "Monthly Weather Review", 1888, p. 235), shown thus (52.4).

The divergent records at Carlsbad, N. Mex., given hereon, are discussed in the Appendix.

The lines of equal evaporation are based on the Evaporimeter records of 1887-88; and are taken from a discussion of these by H. H. Kimball, Librarian, U.S. Weather Bureau, in *Engineering News*, Vol. 53, p. 354.

SKETCH MAP
OF
TEXAS, NEW MEXICO, AND NORTHERN MEXICO
SHOWING
OBSERVED DEPTHS
OF
EVAPORATION
AND
LINES OF EQUAL EVAPORATION
FOR 1887-88.

SCALE OF MILES
100 50 0 50 100 150 200

FIG. 3.

over outflow, and estimating the corresponding depth of the remainder on the lake surface—or the evaporation depth. The data underlying this estimate of the evaporation depth are given in full in the Appendix, where fuller discussions of all the data will be found.

Methods.—The general method used in the Appendix to estimate a safe yearly evaporation depth from Lake Conchos consists of the following principal steps:

1.—The available data of the comparative evaporation depths from pans of different sizes and from floating-pans as compared with land-pans, were considered carefully and made use of—and thus all the Texas and New Mexico evaporation data were corrected to evaporation depths corresponding to those from 3-ft. square floating-pans.

2.—Having thus corrected all the Texas and New Mexico data of evaporation depths to a uniform and comparable basis, the probable evaporation depths from 3-ft. square pans on Lake Conchos were deduced from them by several more or less independent methods, making use of the comparative mean temperatures and elevations of the various evaporation stations.

3.—The observed evaporation depths from 3-ft. square pans floating on Lake Conchos, as measured for short periods (2 and 5 months), were expanded by two different methods to cover a full year.

4.—A comparative relation then was adopted between evaporation depths as measured in 3-ft. square floating-pans and the actual evaporation depths from large reservoir surfaces; and the probable average yearly evaporation depth from Lake Conchos was thus estimated by each of the various methods used in deducing (from the Texas and New Mexico and the La Boquilla data) the yearly evaporation depth from 3-ft. square pans floating on the lake.

5.—The actual evaporation depth from the surface of Lake Conchos was estimated (as closely as practicable) by using the excess of inflow over outflow, plus waters stored for a 7-month period—during which continuous observations had been made of the fluctuating elevations of the lake surface, and continuous measurements of the inflow at the head of the lake, at Pilar de Conchos; during which period there had been practically no inflows (because of almost no rain) between Pilar de Conchos and the La Boquilla Dam; and during which all outflow through the dam (and all losses of stored waters except evaporation losses) had been measured as carefully as practicable.

6.—Finally, from a consideration of the evaporation depth as estimated by the last-named method (which is a full-sized check method) and the evaporation depths as estimated by the other methods, the

adoption was made of a safe average yearly evaporation depth from the surface of Lake Conchos.

This adopted evaporation depth evidently is a gross depth; and, for use in the water-supply investigation, requires to be reduced by the average yearly depth of rainfall on the lake surface.

With the foregoing general explanation of the principal steps comprised in this evaporation investigation, estimations were made of the probable average yearly evaporation depth on Lake Conchos by the following three methods (two of them with two variations):

Method A.—Relations were determined between the monthly evaporation depths (in 3-ft. square floating-pans) and the mean monthly temperatures, at the six evaporation stations in Texas and New Mexico; and (by comparison of the evaporation depths at the six stations of different elevations, all in the Plateau Region) between the evaporation depths at any given temperature, at various elevations above sea level. These relations of evaporation temperature and evaporation altitude were then applied to the observed and corrected evaporation data of Texas and New Mexico, in conjunction with the known elevation of Lake Conchos and its known mean monthly temperatures (which had been observed for a little more than 2 years); and thus the probable mean yearly evaporation depth was estimated, as measured in a 3-ft. square pan floating on the lake.

Method B.—Method *B* was used in two variations, *a* and *b*:

(*a*) From the available evaporation data of Texas and New Mexico, the proportion of a full year's evaporation depth which occurs during the 6 months, January-June, was determined; and from this average proportion and the observed evaporation depths at La Boquilla for the (nearly) 6 months, January-June, 1914, the probable average evaporation depth from the lake for a full year was estimated.

(*b*) From the temperature-evaporation relations of Method *A*, the Texas and New Mexico observed evaporation depths and temperatures, and the known mean monthly temperatures at Lake Conchos during the 6 months, January-June, the corresponding theoretical evaporation depths at the lake during the 6 months were estimated. These theoretical evaporation depths were compared with the evaporation depths actually measured at the lake during the same period, and the percentage relation between theoretical and actual evaporation depths thus obtained was applied to the average annual evaporation

depth previously estimated for the lake by the temperature-evaporation study of Method *A*. The average yearly evaporation depth resulting from Method *A* was thus modified and made more reliable by the use of the nearly 6 months' local evaporation measurements made at the lake.

(A third variation of Method *B* was attempted, using short-period evaporation and temperature observations at El Paso, Tex., made simultaneously with those at Lake Conchos; but this third variation proved to be worthless, because of the shortness of the El Paso record and the apparent vagaries of the temperature there at that time.)

Method C.—Method *C* made use of the estimated excess of inflows into Lake Conchos over outflows, plus waters stored, expressing that excess (or evaporation loss) in terms of depth on the lake surface, and then expanding this estimated gross evaporation depth from the lake surface for the 7-month period into a full yearly period and depth, by the sub-methods, *a* and *b*, described under Method *B*.

This concludes the general description of the methods used.

Results and Conclusions.—The results and conclusions of this evaporation investigation may be divided into two groups, subsidiary and principal, with subjects as follows:

Subjects of Subsidiary Results and Conclusions.

(*a*) Relative evaporation depths from evaporation pans of different sizes;

(*b*) From floating-pans in comparison with land-pans;

(*c*) From large reservoirs in comparison with floating-pans;

(*d*) Relations between evaporation depth and mean temperature;

(*e*) Relations between evaporation depth and elevation above sea level.

The principal result and conclusion is the main subject of inquiry of the investigation—the probable average annual evaporation depth from the surface of Lake Conchos, as estimated by the three (or rather five) more or less independent methods already described.

SUBSIDIARY CONCLUSIONS.

The subsidiary conclusions of the investigation are given in Table 2.

In Table 2, the larger evaporation losses from smaller than from larger pans (shown by (*a*)) and from floating-pans than from large

TABLE 2.—SUBSIDIARY CONCLUSIONS.

(*a*)—Evaporation depth from a 2-ft. square pan	= about	108%	of that from a 3-ft. square pan.	
Evaporation depth from a 2½-ft. square pan	= "	104%	" " " " 3 " " "	
Evaporation depth from a 3-ft. square pan	= "	100%	" " " " 3 " " "	
Evaporation depth from a 4-ft. square pan	= "	93%	" " " " 3 " " "	
Evaporation depth from a 5-ft. square pan	= "	86%	" " " " 3 " " "	
Evaporation depth from a 6-ft. square pan	= "	80%	" " " " 3 " " "	

(*b*)—Evaporation depth from a floating pan = about 80% of that from a land-pan of the same size.

(*c*)—Evaporation depth from a large reservoir = about 62% of that from a 3-ft. square pan floating thereon.

(*d*)—In the Great Plateau Region, and at elevations above sea level of 3 000 to 5 000 ft. and monthly evaporation losses (from 3-ft. square floating-pans) of 2 to 10 in. the increases in the evaporation depth are nearly in direct proportion to the increases in mean monthly temperature: but at lower elevations (3 000 to 600 ft.) the evaporation depth increases with the mean temperature, but much less rapidly than in direct proportion.

(*e*)—In the Great Plateau Region, and at elevations above sea level of 600 to 5 000 ft. and monthly mean temperatures higher than 70° Fahr., the increases in the evaporation depth at any given mean temperature are directly proportional to increases in elevation; but at monthly mean temperatures lower than 70° Fahr., and at the lower elevations, the evaporation depth at any given mean temperature increases with the increase in elevation, but less rapidly than in direct proportion.

reservoirs (shown by (*c*)), presumably are due to the greater heating up of the water in smaller than in larger pans, and in pans than in reservoirs, because of the greater confinement and less circulation of the smaller masses of water, and to the deterrent effect on the evaporation losses of the better and more complete "vapor blankets" existing over larger pans and over reservoirs.

The smaller evaporation losses from floating-pans than from land-pans, shown by (*b*), presumably are due to the greater deterrent effect of the better "vapor blanket" over floating-pans; and to the lower temperature of the water in floating-pans, caused by the influence of the still cooler water in the reservoir.

Subsidiary conclusions (*d*) and (*e*) are formed by inspection from Tables 23, 24, and 25 of the Appendix; and from Fig. 4—which shows by diagrams the relations between evaporation depths and mean temperatures and elevations.

The subsidiary conclusions presented in Table 2 are worked out and discussed in full in the Appendix, where they are shown more completely by tables and diagrams.

PRINCIPAL CONCLUSIONS.

The conclusions as to the average yearly evaporation depth from Lake Conchos, as estimated by the three more or less independent methods and their two variations, are shown in Tables 3 and 4.

The evaporation depths of Tables 3 and 4 are derived and discussed in full in the Appendix.

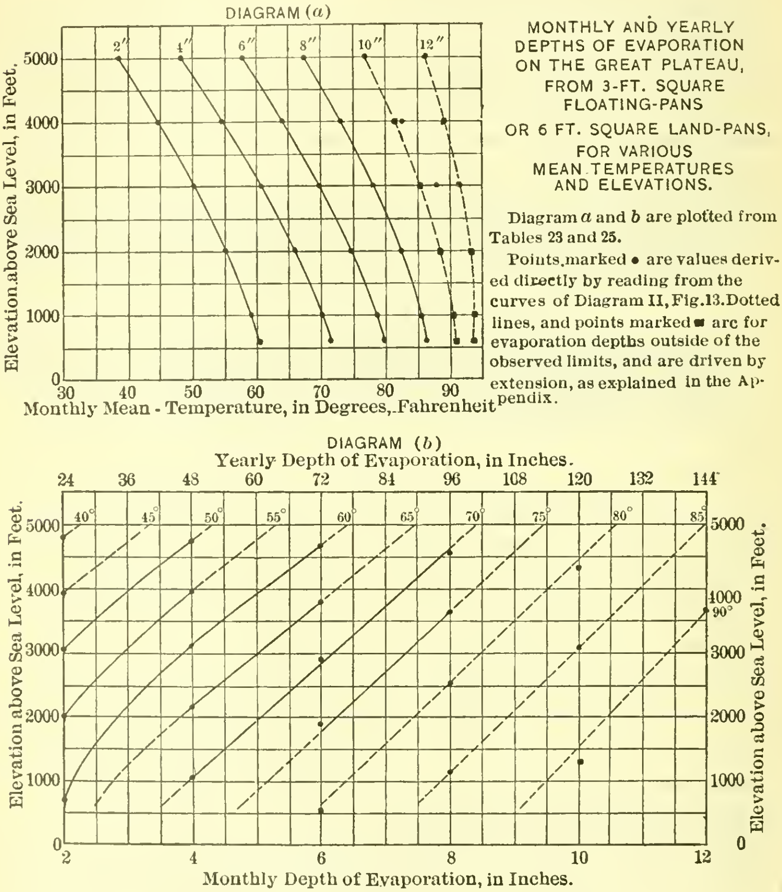


FIG. 4.

It is apparent from the percentage relations of the foregoing summary that the evaporation depth from the surface of Lake Conchos, estimated by Method *A*, is in very close agreement (within 1.7%) with the depth estimated by Method *B*; but that the depths estimated by Method *C* are about 10% greater.

TABLE 3.—FIVE ESTIMATED VALUES OF THE AVERAGE YEARLY EVAPORATION DEPTH FROM THE SURFACE OF LAKE CONCHOS.

By Method A:	Estimating the evaporation from measured yearly pan evaporations in Texas and New Mexico, modified for differences in mean temperature and in elevation	52.5 in.
By Method B_a:	Estimating the evaporation from the 6 months' (nearly) measurements of pan evaporation at Lake Conchos; expanded to a full year by the percentage relation adopted (from the Texas and New Mexico evaporation data) between the evaporation depth for those 6 months and that for a full year	54.1 in.
By Method B_b:	Same as B_a, except that the observed depth of pan evaporation at La Boquilla for 6 months is expanded to a full year's depth by evaporation-temperature relations	53.4 in.
By Method C_a:	Estimating the evaporation from the excess of the inflows over the outflows plus the increase in storage of the lake for 7 months; expanded into a full year's evaporation depth by the adopted percentage relation borne by those 7 months' evaporation depth to that of a full year (as in Method B_a)	57.7 in.
By Method C_b:	Same as Method C_a, except that the evaporation depth for the 7-month period is expanded to that for a full year by the use of evaporation-temperature relations (as in Method B_b)	59.3 in.

TABLE 4.—SUMMARY AND COMPARISON OF THE FIVE VALUES OF YEARLY EVAPORATION FROM LAKE CONCHOS.

Method of estimating.	Average yearly gross evaporation depth from the lake.
A	52.5 in. = 100.0%
B_a	54.1 " = 103.0%
B_b	53.4 " = 101.7%
Mean of B_a and B_b	53.75 " = 102.4%
Mean of A and Mean B	53.13 " = 101.2%
C_a	57.7 " = 109.9%
C_b	59.3 " = 113.0%
Mean of C_a and C_b	58.50 " = 111.5%
Mean of A, Mean B, and Mean C	54.92 " = 104.6%
Mean of A, B_a, B_b, C_a and C_b	55.40 " = 105.5%
Adopted value	55 0 in. ≐ 104.8%

The evaporation depth resulting from Method B is more reliable and accurate than that from Method A, because the former includes the consideration of, not only all the data used in Method A, but also the local pan observations of evaporation made at Lake Conchos. For reasons about to be given, the evaporation depths resulting from Method C are known to be somewhat greater than the true depths, and indeed are regarded rather as reasonably close (and full-sized

model) rough checks on the results of Method *B*, than as equally careful determinations of the evaporation depth by another method.

Hence it is believed that the most reliable and accurate value of the average yearly evaporation from the surface of the lake is the depth of 53.75 in. (1.365 m.) resulting from Method *B*.

The principal reasons for believing the results of Method *C* to be greater than the true value of the evaporation depth are as follows:

(1)—During the 3 months from the middle of June to the middle of September, 1913, the surface of Lake Conchos was raised 100 ft. by the flood waters impounded, from a maximum depth of 29.5 ft. to one of 129.5 ft.; and its surface area was increased about 15½ sq. miles, from less than 0.4 to about 15.8 sq. miles. The area and elevation of the lake then remained nearly constant until the beginning of the next flood season, in June, 1914. Method *C* is based on a consideration of 7 months of this period, from October, 1913, to April, 1914, inclusive.

Even though the bottom of the reservoir or lake is to a large extent rocky and apparently but slightly absorptive of the water (see Fig. 2), during the 7 months, October to April, there must nevertheless have been some water lost by absorption in the 15½ sq. miles of reservoir bottom inundated recently and for the first time. This obviously is true of the soil-covered portions, and must be true of the bare rock portions also, because the seepage springs fed by the lake (most of which springs made their first appearance during these 7 months, October to April) are known to reach, in their course from lake to springs, depths of at least 2 000 ft. below the lake surface. In Method *C*, however, no attempt was made to estimate separately such absorbed water, which in consequence was treated as if part of the water evaporated. Hence, for this reason (and for some others also), the evaporation depths resulting from Method *C* are known to be greater than the actual ones.

It is apparent from Table 4 that the yearly evaporation depth by Method *C* exceeds that by Method *B* by only (58.50 — 53.75) = 4.75 in. This excess, with an average void content of the material forming the reservoir bottom of, say, 20%, would require an average depth below the reservoir bottom of only 24 in. for its entire absorption. Again, during the 7 months, October to April, the lake remained nearly constant at a level of about 1 289 m., an area of about 16.2

sq. miles, and about 346 000 acre-ft. of impounded water. The excess depth of 4.75 in. over the 16.2 sq. miles equals about 4 100 acre-ft., or only 1.2% of the impounded water. It has been said by Fteley and Herschel (referring presumably to some of the reservoirs of the New Croton Aqueduct system) that some reservoirs on being emptied return from ground storage as much as 10% of their capacity.*

It is evident from either view of the question that there is no improbability in the assumption that 4.75 in. depth of water, assumed by Method *C* as part of the water evaporated, instead was absorbed by the bottom of the reservoir during the 7 months considered.

(2)—During the 7-month period used in Method *C*, efforts were made to measure all seepage losses from the lake. Certain losses were unmeasurable, however, such as ground evaporations from wet soil between the springs and the measuring weirs, and such seepages as may enter the river below its water surface. In an attempt to cover such unmeasurable seepages, 10% was added arbitrarily to the sum of the measured seepages; but it is not unlikely that the 10% so assumed may be too little.

(3)—At lake level 1 289 m., the area of the lake was about 16.2 sq. miles and its shore line about 86.4 miles, or its equivalent mean diameter or width about $[(16.2 \div 86.4) \times 4] = 0.75$ mile. At the average future lake level of 1 312.5 m., the area will be about 55.3 sq. miles, the shore line about 152 miles, and the equivalent mean width about 1.45 miles, or nearly twice as great. It may be that the greater width in the future will give a better "vapor blanket", and one more deterrent to evaporation losses—and hence that the yearly evaporation depth throughout the future may be less than that estimated from the 7 months, October to April, used in Method *C*. Such smaller evaporation depth because of the larger lake surface is regarded as very doubtful, however.

(4)—In addition to the evaporation from the lake surface, there always will be some evaporation from the narrow strip along the shore which soaks up water by capillary action. During the 7 months considered in Method *C*, however, with the lake level nearly constant at about 1 289 m., such evaporation losses were relatively much greater

* However, part of the 10% presumably was from the raised ground-water outside the horizontal limits of the water stored in the reservoir.

than they will be throughout the future, because at 1 289 m. there are about 5.3 miles of shore line per square mile of water surface, and at the average lake level of 1 312.5 m. there will be only about 2.7 miles per square mile, or only 51% as much.

For the several reasons given, it is believed that the somewhat greater evaporation depths resulting from Method *C* may be disregarded, at least partly; and that a mean of the five values of evaporation depth obtained by the various methods may be adopted with safety as being not too low. Hence the mean of the five values, say, 55 in., or 1.40 m., was adopted as a safe mean annual evaporation depth from the surface of Lake Conchos throughout the future. This 55 in. is a gross annual evaporation depth, and in use will be reduced by the depth of rain falling directly on the reservoir surface, which amounts to an average of about 12.0 in. per year.

It should be added that the foregoing conclusion is based on all the La Boquilla or Lake Conchos evaporation data secured up to the beginning of the heavy floods of June, 1914; but that the 55 in. thus estimated from the use of all the data secured is in practically exact agreement with the results estimated from the shorter period of 2 months, beginning in January and ending in March, 1914. This close agreement is shown by the comparison in Table 5.

The evaporation observations at the lake were terminated early in June, 1914, by heavy storms and floods, which swamped the floating-pans, made the inflows to the lake unmeasurable, etc.

From an evaporation-temperature study (given in the Appendix), it was ascertained that the monthly percentage distribution of the year's gross evaporation depth is about the same at the lake as at the evaporation stations in Texas and New Mexico used in this investigation; and the probable percentages of the yearly evaporation occurring each month thus were adopted. These adopted monthly percentages and their corresponding monthly evaporation depths in inches are given in Table 6. Also (as of interest in showing the characteristics of the local climate), there are included in that table the average depth of rain falling on the reservoir surface each month and the corresponding net evaporation depth.

It is worthy of repetition here that in 1911 Mr. John R. Freeman, adopted 55 in. as the probable average yearly gross evaporation

TABLE 5.—COMPARISON OF VALUES FOR PERIODS OF TWO AND FIVE MONTHS.

Method.	Yearly Gross Evaporation Depth from Lake Conchos.			
	From 2 months' pan observations at Lake Conchos.		From 5 months' pan observations at Lake Conchos.	
	Inches.	Percentage.	Inches.	Percentage.
A	52.5*	= 100.0	52.5*	= 100
B_a	52.9†	= 97.9	54.1†	= 100
B_b	52.4†	= 98.2	53.4†	= 100
C_a	55.4‡	= 96.2	57.7‡	= 100
C_b	56.2‡	= 94.8	59.3‡	= 100
Mean	53.88	= 97.3	55.40	= 100
Adopted	55.0	= 100.0	55.0	= 100

* Based on no pan evaporations at the lake, but on more than a year's mean monthly temperatures there.

† Based on Method A, as modified by 2 months' and 5 months' pan evaporations at the lake.

‡ Based on 4 months' and 7 months' lake observations, expanded into a full year by Methods B_a and B_b, respectively.

TABLE 6.—ADOPTED AVERAGE MONTHLY GROSS AND NET EVAPORATION DEPTHS FROM LAKE CONCHOS.

Month.	Gross Evaporation Depths.		Rain Falling Directly on Lake Surface.		Net Evaporation Depths from Lake Surface.	
	Percentage.	Inches.	Percentage.	Inches.	Inches.	Percentage.
(1)	(2)	(3)	(4)	(5)	(6) (3—5)	(7)
Jan	4.8	= 2.6	1.0	= 0.1	2.5	= 5.8
Feb	5.6	= 3.1	2.5	= 0.3	2.8	= 6.5
Mar	7.4	= 4.1	2.5	= 0.3	3.8	= 8.8
Apr	9.4	= 5.2	3.0	= 0.4	4.8	= 11.1
May	11.5	= 6.3	4.0	= 0.5	5.8	= 13.5
June	11.7	= 6.4	13.7	= 1.7	4.7	= 10.9
July	12.2	= 6.7	17.7	= 2.1	4.6	= 10.7
Aug	11.1	= 6.1	21.5	= 2.6	3.5	= 8.2
Sept	9.4	= 5.2	20.2	= 2.4	2.8	= 6.5
Oct	7.7	= 4.2	7.7	= 0.9	3.3	= 7.7
Nov	5.1	= 2.8	3.4	= 0.4	2.4	= 5.6
Dec	4.1	= 2.3	2.8	= 0.3	2.0	= 4.7
Year	100.0	= 55.0 or 1.40 m.	100.0	= 12.0 or = 0.305 m.	43.0 or 1.09 m.	= 100.0

depth from Lake Conchos, and this without special study or investigation, or local observations, but merely by the exercise of judgment.

This contribution to the existing engineering information of the evaporation losses from reservoirs is offered with a full realization that many other meteorological conditions than mean temperatures and elevations have more or less influence on the depth of evaporation; and that whenever time and opportunity permit of their accumulation, data of humidities, wind velocities and frequencies, etc., also are desirable as aids in estimating probable future evaporation losses.

However, it is believed to be apparent as the result of this evaporation study that whenever time or opportunity is lacking for the collection of full local meteorological data (as so frequently is the case in engineering investigations), then reliable conclusions as to monthly and yearly evaporation depths from reservoirs may be drawn without other data than observed pan evaporations elsewhere (at places having similar characteristics of climate) modified skilfully by the comparative mean temperatures and elevations above sea level.

Also, it is believed that for places within the limits of the Great Plateau, from Mexico on the south to perhaps Colorado and Utah on the north, fair general approximations to local monthly and yearly evaporation depths may be read directly from Fig. 4 without other data than local mean temperatures and elevations.

The data on which this study is based, and the working out of this investigation, are given in full in the Appendix.

APPENDIX

Evaporation Pans at Lake Conchos.—The locations and exposures of the pans are explained sufficiently in Table 34 (Plate XXXV). The depths of water in the pans were read frequently (part of the time daily), by an ordinary rule, to $\frac{1}{32}$ in. As far as practicable, the pans were kept filled with water to a depth of about 10 in., or about 2 in. below their rims. The depths were corrected for the infrequent rainfalls by near-by rain gauges.

Subsidiary Conclusion (*a*):

Relative Evaporation Depths from Pans of Different Sizes.

It has been noted that when evaporation measurements are made in pans, the depths evaporated are greater, under the same climatic conditions, if the measurements are made in small than in large pans.

Experiments to determine the relative evaporation depths from pans of different sizes have been made by the U. S. Weather Bureau.*

Professor Bigelow states that the evaporation depths from pans are given by the formula:

$$E_o = C_2 \times \frac{e_s}{e_d} \times \frac{d_e}{d_s} \times (1 + 0.070\, w)$$

in which

E_o = the evaporation-depth in 4 hours;
e_s = the vapor pressure at the water temperature;
e_d = the vapor pressure at the dew-point temperature;
$\frac{d_e}{d_s}$ = the coefficient from the physical table;
w = the wind velocity, in kilometers per hour;
and C_2 = a variable, depending on the size of the evaporation pan.

He states, further, that, based on experiments made by the U. S. Weather Bureau at the Salton Sea and elsewhere, various values of C_2 are as follows:

For 2-ft. square pans	$C_2 = 0.042$
" 4- " " "	" $= 0.036$
" 6- " " "	" $= 0.031$
" large lake surfaces	" $= 0.024$

From these values of C_2, the curve of Fig. 5 is plotted; and from that curve Table 7 is made up.

* The conclusions therefrom, as summarized by Frank H. Bigelow, Professor of Meteorology, U. S. Weather Bureau, in charge of the Climatological Division, are published in *Engineering News*, Vol. 63, pp. 694–5.

TABLE 7.—RELATIVE EVAPORATION DEPTHS FROM PANS OF VARIOUS SIZES.

Size of pan.	Coefficient, C_2.	Relative evaporation depth.
2 ft. square.......	0.0420	108% of the depth from a pan 3 ft. square.
2½ " "	0.0405*	†104% " " " " " " 3 " "
3 " "	0.0389*	100% " " " " " " 3 " "
4 " "	0.0360	93% " " " " " " 3 " "
5 " "	0.0333*	86% " " " " " " 3 " "
6 " "	0.0310	80% " " " " " " 3 " "

* Interpolated from the curve of Fig. 5.

† By a clerical error, this was used as 105% in the succeeding parts of the investigation—in which this error has been left uncorrected because unimportant.

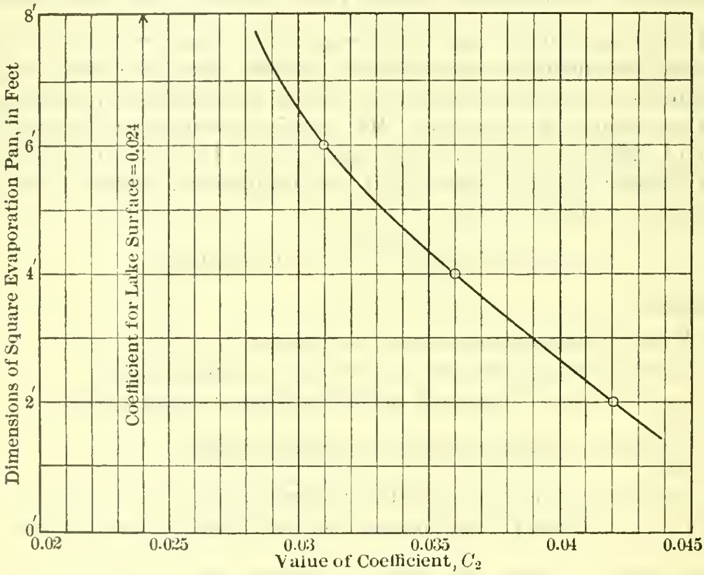


FIG. 5.

In a full consideration of the effect of the size of the pan on the depth of evaporation, it is evident that, not only the horizontal dimensions of the pans should be considered, but also the depths. However, available records of evaporation tests, though usually stating the horizontal dimensions of the pans, more often than not fail to mention their depths; and the depths of most of the pans used in the Texas and New Mexico evaporation tests which form the basis of this study are unknown. Hence a consideration in this study of the effect of depth of pan on evaporation depth necessarily had to be omitted.

The standard evaporation pan of the U. S. Geological Survey is believed to be 3 ft. square and 18 in. deep; and during evaporation tests it is supposed to be kept filled with water to within about 3 or

4 in. of the rim. The Texas and New Mexico pans are believed to have been generally less than 18 in. deep. The pans used for the Lake Conchos evaporation tests were 3 ft. square and only 12 in. deep; and they were kept filled with water to approximately 2 in. below the rim. It now seems that it might have been preferable if they had been 18 in. deep, the standard depth of pan of the U. S. Geological Survey.

However, the purposes of the Rio Conchos investigation were to estimate an evaporation loss which would be certainly not below the actual loss, and (by deducting the evaporation loss from the gross stream flow) to estimate a net stream flow available for power production which would be certainly not greater than the actual; and for these purposes pans 12 in. deep probably were safer and more conservative than deeper ones would have been, as it is believed that shallower pans give greater evaporation losses than deeper ones. Also, the 12-in. depth is believed to be more nearly in agreement with that of the Texas and New Mexico pans, from which the evaporation depth at Lake Conchos was being deduced by modifications for differences in mean temperatures and in elevations.

Subsidiary Conclusion (*b*):

Relative Evaporation Depths from Land-Pans and from Near-By Floating-Pans.

It has been found by observation that the depth of evaporation from floating-pans is about 80% of that from near-by land-pans of the same size. A few instances showing this relation are as follows:

*Granite Reef, Ariz.**—(Presumably at least a year's observations.)

A year's evaporation from 4-ft. square, land-pan = 115.18 in. = 100%
" " " " 4-ft. " floating-pan = 97.74 in. = 84.8%

*California, Ohio.**—(Presumably at least a year's observations.)

A year's evaporation from 3-ft. square, land-pan = 61.83 in. = 100%
" " " " 4-ft. " floating-pan = 45.99 in. = ——
" " " " 3-ft. " floating-pan (equals 100% ÷ 93% = 107.5% of that from a 4-ft. square, floating-pan) = 49.4 in. = 80.0%

Coyote, Cal. (Near San José).—These observations were made under the writers' supervision, and consist of the evaporations from two pans floating on the Laguna Seca (a shallow natural lake), as compared with the evaporations from three near-by land-pans. These observations covered 2 years.† The briefest summary of the Laguna Seca evaporation observations is as follows:

* *Engineering News*, Vol. 63, p. 695.

† Abstracts of these observations (with additional evaporation data) are published in *Engineering News*, Vol. 67, pp. 380–83.

	1904.	1905.
A year's evaporation depth from a 3-ft. square, land-pan (mean of three pans)......	56.6 in. = 100%	56.9 in. = 100%
A year's evaporation depth from a 3-ft. square, floating-pan (mean of two pans)........	41.2 in. = 72.8%	48.3 in. = 85.0%

Mean of the 2 years: floating-pan = 78.9% of land-pan.

*Salton Sea, Cal.**—(Presumably at least a year.)

A year's evaporation depth from a 2-ft. square, land-pan.......................	= 164.50 in. = ——
A year's evaporation depth from a 4-ft. square, land-pan (equals 93 ÷ 108 = 86.1% of that from a 2-ft. square land-pan.)	= 141.6 in. = 100%
A year's evaporation depth from a 4-ft. square, floating-pan (mean of two pans, in tower 2 ft. above surface of sea)....	= 107.55 in. = 76.0%

TABLE 8.—SUMMARY OF EVAPORATION TESTS.

Evaporation station.	Evaporation depth from a floating-pan, as a percentage of that from a near-by land-pan of the same size.
Granite Reef, Ariz..............................	84.8%
California, Ohio..............................	80.0%
Coyote, Cal..............................	78.9%
Salton Sea, Cal..............................	76.0%
Mean of the four..............................	79.9%

Based on the measured instances in Table 8 (and especially on the careful Coyote tests made under the writers' own supervision, and comprising the records of two floating-pans for 2 years) the depth of evaporation from a floating-pan was adopted as 80% of that from a near-by land-pan of the same size. The 80% relation appeared to be confirmed, also, by values then worked up from the Lake Conchos tests, as shown in Table 9.

TABLE 9.—PARTIAL LAKE CONCHOS TESTS.

Periods: (1 and 2 months).	Evaporation from:	
	Land-pan (Pan No. 1).	Floating-pan (Pan No. 2).
After 1 month's evaporation (to middle of February, 1914)...	100%	79.7%
" 2 months' " (" " " March, ")...	100%	80.1%

* *Engineering News*, Vol. 63, p. 695.

However, the remaining portion of the Lake Conchos tests (which was not examined as to the 80% relation until the present time, in connection with the writing of this paper) shows a disagreement with the 80% relation, as may be seen in Table 10.

TABLE 10.—PARTIAL LAKE CONCHOS TESTS.

Periods: (1 and 2 months).	Evaporation from: Land-pan (Pan No. 3).	Floating-pan (Pan No. 4).
After 1 month's evaporation (at middle of February, 1914)...	100%	85.5%
" 2 months' " (" " " March, ")...	100%	87.0%

(Pan No. 4 then was discontinued because it was very frequently swamped by waves.)

TABLE 11.—PARTIAL LAKE CONCHOS TESTS.

Periods: (3, 4, and 5 months).	Evaporation from: Land-pan (Pan No. 1).	Floating-pan (Pan No. 2).
After 3 months' evaporation (at middle of April, 1914)......	100%	84.8%
" 4 " " (" " " May, ")......	100%	87.8%
" 5 " " (" " " June, ")......	100%	86.5%

As mentioned already, the disagreements at La Boquilla with the 80% relation were not realized until the present time. It is believed, however, because of the much longer tests made for this relation at Coyote and elsewhere, that the adoption of the 80% relation still is justified.

Subsidiary Conclusion (*c*):

Relative Evaporation-Depth from Large Reservoirs, as Compared with That from 3-Ft. Square Pans Floating Thereon.

For some time past it has been recognized more or less generally that the evaporation losses from large reservoirs usually have been over-estimated, and that such losses are materially less than the evaporation depths measured in pans, even when the pans are floating on the reservoir.

In 1907 Professor Bigelow discussed this subject as follows:

"Large bodies of water are differently affected by the wind from small bodies, such as can be experimented with in the shape of tanks and pans. As the air moves across a reservoir and gradually becomes charged with moisture, its rate of absorption diminishes, and the average rate of evaporation for a broad surface is therefore less than

for small surfaces. For this reason, the formula for evaporation cannot be put on a sound basis, without taking account of large water surfaces as well as of small. The conditions afforded by the Salton Sea are peculiarly suitable for the investigation of the laws of evaporation."

Professor Bigelow reached the following conclusions*:

"* * * in the arid regions of the West, it seems probable that a lake or reservoir evaporates about five-eights as fast as an isolated pan placed outside of the vapor blanket; in other words, this vapor blanket seems to conserve about three-eights of the water that would otherwise be lost by the evaporation. It is important that similar experiments with towers be made in the central and eastern portions of the United States, in the prevailing damp climates, to discover whether similar rules can be applied in practise. A careful campaign on the theory of evaporation is evidently demanded to elucidate this complex function of the evaporation of water in the open air, and it is probable that several years will be required in order to bring it to a satisfactory conclusion."

Again, in 1910, in an article† by Professor Bigelow, his conclusions in this matter are given. At the close of that article he states as follows:

"These data [the Salton Sea evaporation tests] indicate a nearly uniform rate of evaporation over the area of the water, beginning a short distance from the shore. If 70 ins. is admitted as the amount evaporated from the Salton Sea,‡ there remains 38 [38.65?] ins. as the difference between the [evaporation from the] water in the sea and that in the lower swinging pan§. It was not possible to float pans in the waves of the Salton Sea, and it is not easy to take the step in the research just indicated in order to arrive at the true coefficient [of evaporation] for the water of the Salton Sea. Our computations on the temperatures are not yet completed regarding this point, and we are conducting another experiment‖ with a series of pans 2 ft., 4 ft., 6 ft. and 12 ft. in diameter. By computing with the formula,¶ assuming values of $C_2 = 0.021, 0.023, 0.025, 0.027, 0.029$, and using the observed temperatures of the Salton Sea surface water, it is now probable that the coefficient for a large water surface is $C_2 = 0.024$."

Professor Bigelow states elsewhere in the same article:

"The automatic record indicates that the Salton Sea has fallen about 4.60 ft. annually. * * * It is probable that the inflow from the New and Alamo rivers, together with the precipitation flowing in from the surrounding country, amounts to something like 1.5 ft. [annually]. [Hence,] allowing for the inflow, we have a little more or less than 6.0 ft. by evaporation [annually from the Salton Sea]".

* "Monthly Weather Review", 1908.

† *Engineering News*, Vol. 63, pp. 694–695; this being the article which already has been referred to in discussing the relative depths of evaporation from pans of different sizes, and from land-pans as compared with floating-pans.

‡ See additional quotation following this one.

§ Two feet above the surface of the sea.

‖ Which it is believed was discontinued before completion.

¶ Given on p. 1741.

From Fig. 5 and Table 7, the value of C_2 for a 3-ft. square pan* is about 0.0389. This value, compared with the value of 0.024, gives the depth of evaporation annually from a large water surface as $(0.024 \div 0.0389) = 61.7\%$ of that from a 3-ft. square pan floating thereon.

In an extensive table in the foregoing article Professor Bigelow gives many data of the Salton Sea evaporation tests, among them those in Table 12.

TABLE 12.—SALTON SEA TESTS.

Size and location of pan.	TOWER NO. 2, 500 FT. AT SEA.		TOWER NO. 4, 7 500 FT. AT SEA.	
	Inches.	Percentage.	Inches.	Percentage.
Annual evaporation depth from 4-ft. square* pan, 2 ft. above surface of sea....................	108.65	100.0	106.45	100.
Annual evaporation depth from 4-ft. square* pan, 45 ft. above surface of sea....................	137.71	126.7	140.02	131.6

* Pan mentioned as 4 ft. "in diameter", but believed to have been 4 ft. "square", instead.

From Table 7, the evaporation depth from a 3-ft. square pan is $(0.0389 \div 0.0360) = 108.0\%$ of that from a pan 4 ft. square, hence, if the lower pans of Table 12 had been only 3 ft. square, their evaporation depths should have been:

Lower pan in Tower No. 2 = $(108.65 \times 1.08) = 117.5$ in.
" " " " No. 4 = $(106.45 \times 1.08) = 115.0$ "

From the 70 in. believed by Professor Bigelow to have been the annual evaporation depth from the surface of the Salton Sea, and the assumption that the evaporation depths from pans 2 ft. above the surface of the sea were the same as if the pans had been floating in the sea instead, the following relations result:

Evaporation depth from the surface of Salton Sea, as a percentage of that from a pan 3 ft. square floating thereon .... $\left\{ \begin{array}{l} \text{From Tower No. 2} = (70 \div 117.5) = 59.6\% \\ \text{From Tower No. 4} = (70 \div 115.0) = 60.9\% \\ \text{Mean} = \ldots\ldots\ldots\ldots\ldots\ 60.3\% \end{array} \right.$

This value varies but little from the 61.7% derived from the $C_2 = 0.024$ adopted tentatively by Professor Bigelow, presumably from a fuller and more general consideration, based on more complete data.

* Presumably, from the context, a floating-pan.

The following quotations show that there has been some general recognition in the recent past of the smaller evaporation depths from reservoirs than from pans, and from large than from small pans. All the opinions quoted seem to be based on the Salton Sea evaporation tests.

Mansfield Merriman, M. Am. Soc. C. E., states:*

"The evaporation from a pan† 2 feet in diameter‡ is about 75%, that from a pan 4 feet in diameter is about 50%, and that from a pan 6 feet in diameter is about 30% greater than the evaporation from a large pond or lake."

By the comparison in Table 13, the foregoing relations are seen to be identical with those of Table 7.

TABLE 13.—COMPARATIVE EVAPORATION DEPTHS FROM PANS OF VARIOUS SIZES.

Size of pan.	Comparative percentages.
2 ft. square	175% = 108%
3 " "	*162 = 100
4 " "	150 = 93
6 " "	130 = 80
Lake surface	100 = †62

* From Table 7, the evaporation from a 3-ft. pan = 100/80ths of that from a 6-ft. pan and 130% × 1.25 = 162 per cent.

† Identical with Professor Bigelow's value.

F. T. Robson, Assoc. M. Am. Soc. C. E., concludes,§ after a careful personal check and revision of the Salton Sea data, that the average evaporation depth from the Salton Sea during the 6 years, April 1st, 1907, to April 1st, 1913, was 67.02 in. per year. For the single year, June 1st, 1909, to June 1st, 1910, he accepts the evaporation depth from 6-ft. land-pans as 114.05 in. This value is the mean of the observed evaporation losses from four pans placed outside the "vapor blanket" but otherwise under the same desert conditions as the Salton Sea. These pans were north, east, and south of the sea, at distances of from 4 to 20 miles from its shore line. From these data, Mr. Robson estimates that the evaporation depth from the surface of the sea is (67.02 ÷ 114.055) = 0.588%, or about 59% of that from 6-ft. square land-pans in that vicinity.

Since the evaporation from a 3-ft. pan. is (100 ÷ 80) = 125% of that from a 6-ft. pan, and the evaporation from a floating-pan is 80% of that from a land-pan of the same size, the evaporation from a 3-ft.

* "American Civil Engineers' Pocket-Book" (Second Edition, 1912), p. 1256.

† Presumably a floating-pan.

‡ Presumably 2 ft. "square" instead of "in diameter".

§ *Transactions*, Am. Soc. C. E., Vol. LXXVI (December, 1913), pp. 1516–24.

square floating-pan will be (125% × 0.80) = 100%, the same as that from a 6-ft. square land-pan; hence, in Mr. Robson's opinion, the evaporation from the surface of the Salton Sea was about 59% of that from a 3-ft. square pan floating thereon.

C. E. Grunsky, M. Am. Soc. C. E., concludes,* from a careful personal revision of the Salton Sea data, that the evaporation from the sea during the year, April 1st, 1907, to April 1st, 1908, was 6.14 ft., or 73.7 in. Combining this value with the evaporation of the year, 1909-10, from 6-ft. square land-pans (though such a combination of two different years is not strictly permissible, and is not made by Mr. Grunsky), the evaporation from the Salton Sea itself is approximately (73.7 ÷ 114.05) = 64.6% of that from 6-ft. square land-pans situated near by, or 3-ft. square pans floating thereon.

The several values found for the proportional evaporation from the Salton Sea itself and from 3-ft. square pans floating thereon are summarized in Table 14 (neglecting the 62% quoted from Merriman, which presumably is only Professor Bigelow's conclusion).

TABLE 14.—Evaporation from Salton Sea, as a Percentage of that from 3-Ft. Square Floating-Pans.

Authority.	Yearly Evaporation Depth, from:		Evaporation depth from Salton Sea, as a proportion of that of 3-ft. floating-pans.
	Salton Sea.	6-ft. land-pan or 3-ft. floating-pan.	
Bigelow..........	70 in.*§	*114.05 in.	61.4%
"	69 *§	"	60.5
Robson..........	67.02 †	"	58.8
Grunsky..........	73.7 ‡	"	64.6
Mean of all four values..........			61.3%
Mean of Bigelow's two values..........			60.95
Mean of Robson's and Grunsky's..........			61.7

* For the year, June 1st, 1909, to June 1st, 1910.

† Mean of 6 years, April 1st, 1907, to April 1st, 1913.

‡ For the year, April 1st, 1907, to April 1st, 1908.

§ Used by Professor Bigelow as 70 in.; stated by F. T. Robson as more accurately 69 in. from Professor Bigelow's data.

Mr. Robson states, as one of the conclusions of Professor Bigelow, that tests showed the pan evaporation from somewhat concentrated and brackish Salton Sea water to be 2% less than that from fresh water. However, the pan evaporations of Table 14 presumably are from fresh water, and the Salton Sea during 1907-13 presumably was very little brackish, and much less so than the concentrated sea water in the pans found to have 2% less evaporation than fresh water. Hence it is believed that the proportions in Table 14 (58.8%-64.6%)

* *Engineering News*, Vol. 60 (July–December, 1908), pp. 163–66.

do not require such 2% increases in applying them to reservoirs of fresh water elsewhere.

Of the four values in Table 14, only those of Professor Bigelow are derived from a comparison of the evaporation from the Salton Sea with that from 6-ft. land-pans or 3-ft. floating-pans for the same year; hence it is felt that Professor Bigelow's conclusions are more to be relied on than either Mr. Robson's or that derived from Mr. Grunsky's study.

For that reason, and also because Professor Bigelow's conclusion is based on a study of the values of the coefficient, C_2, his conclusion is that adopted. Hence his value of 62% (being in round numbers his 61.7% corresponding to his adopted value 0.024 of C_2) is that adopted and used throughout the remainder of this investigation.

The adopted conclusion of 62% appears to be based almost entirely on the experiments at the Salton Sea. Hence, for the specific purpose of this general investigation, a comparison seems necessary between the conditions of exposure, etc., existing at the Salton Sea and those at Lake Conchos.

The Salton Sea is regular and somewhat oval in form, and, at the time of the evaporation experiments, was about 45 miles long and from 10 to 16 miles broad, with its surface about 200 ft. below ocean or sea level. It had an area of about 430 sq. miles and about 115 miles of shore line or perimeter. Its maximum depth was about 75 ft. and its average depth about 30 ft. It lies in the trough of a shallow basin, and is open to the winds from every direction.

At spillway level, Lake Conchos will occupy a length, east and west, of about 24 miles, with a surface area of 67.7 sq. miles and 165 miles of shore line. Its shore lines are very irregular (as shown by Figs. 1 and 2), and it has a length on its broken easterly and westerly axis of approximately 30 miles. Its widths vary greatly: at spillway level from about ⅓ mile to about 3⅝ miles, with an average width of perhaps 2¼ miles. Its depths will be about 223 ft. maximum and about 61 ft. average below spillway level. At the average elevation of the lake surface during the full operation of the power-plant (1 312.5 m.), the average depth will be about 55 ft. and the maximum about 208 ft.; and during the 7 months from October, 1913, to April, 1914, they were about 32 and 131 ft., respectively.

As shown by Fig. 2 (a panoramic photograph), Lake Conchos occupies the somewhat canyon-like bed of the Rio Conchos, and in general its shores are steep and precipitous. Because of the adjacent hills, the steep shores, the many points projecting into the lake, and the consequent great variation in width, it is believed that the winds have much less free access to its surface than to that of the Salton Sea.

The principal reason for the smaller evaporation depth from large than from small water surfaces is believed to be the existence of

thicker and more unbroken "vapor blankets" over the larger surfaces, with less frequent changes of the overlying air in contact with the water by new dry and non-humid air. Also, because of its slower heating up, it is believed that there should be less evaporation depth from deep than from shallow reservoirs.

The quotient of a lake surface area divided by its perimeter or shore line is somewhat analogous to the "hydraulic radius" or semi-radius of a circular conduit; and, for comparative purposes, four times such quotients may be regarded as the equivalent mean diameters or widths of lakes. Such mean widths of the Salton Sea and of Lake Conchos, and also their average depths, are compared in Table 15.

TABLE 15.—Comparative Mean Widths and Mean Depths of Salton Sea and of Lake Conchos.

Lake and stage.	Area, in square miles.	Shore line, in miles.	Mean width, in miles.	Mean depth, in feet.
Salton Sea	430	115	15.0 = 100%	30 = 100%
Lake Conchos				
Elev. *1312.5 m	55.3	152	1.46 = 9.7%	55 = 184%
" †1289. "	16.2	86.4	0.75 = 5.0%	32 = 107%

* Average lake level during future operation.
† Lake level during evaporation tests of October, 1913, to April, 1914.

Hence, during future operations, Lake Conchos will have about one-tenth as great a mean width and nearly twice as great a mean depth as that of Salton Sea during the evaporation tests at that sea; while, during the evaporation tests at Lake Conchos, its mean width was about one-twentieth of that of Salton Sea and its mean depth about the same.

With the information at hand, it is impossible to form any opinion of the comparative protective effects of their "vapor blankets" in these two instances. However, at least it is probable that the greater protection of Lake Conchos and its "vapor blanket" from the disturbing effect of winds because of its surrounding ridges, etc., will wholly or partly offset the less protection because of its smaller mean width and proportionally longer shore line. Also, the greater mean depth probably will tend to diminish the evaporation losses from Lake Conchos in the future, below their value during the 7 months, October, 1913, to April, 1914.

With regard only to differences in evaporation depths from large pans and from reservoirs and lakes, it seems probable, from Table 16 and a brief study of its values, on logarithmic co-ordinate paper, that the value of C_2 becomes, for practical purposes, constant and equal to 0.024 when pans reach a size as great as from 12 to 25 ft. square.

TABLE 16.—(FROM TABLE 7 AND FIG. 5.)

Size of pan.	Value of C_2.	Decrease in C_2 for 2 ft. increase in pan.
2 ft. square............	0.042	
		0.006
4 " "	0.036	
		0.005
6 " "	0.031	
		0.003±
8 " "	0.028±	
		(0.004±)*
Lake surface.........	0.024	

* Decrease from 8-ft. pan to "lake surface".

From Table 16 and its logarithmic study, it appears at least very doubtful if there is any material variation in the values of C_2 as between smaller or larger large reservoirs and lakes.

Hence, in the absence of fuller information, the conclusion seemed justified that the relation established for the Salton Sea between the depth of evaporation from reservoirs and that from pans floating thereon, may be applied unchanged to Lake Conchos with a fair degree of confidence in its essential correctness.

Therefore, the yearly evaporation depth from the surface of Lake Conchos was adopted as 62% of the evaporation depth from a 3-ft. square pan floating thereon or from a 6-ft. square land-pan near-by.

Subsequently, in working up by Method *C* the evaporation depth from Lake Conchos, data were acquired permitting of a rough check from the evaporation tests at that lake of this adopted value of 62 per cent. This check is given in full hereinafter—and it will be stated here only that the value thus found at Lake Conchos was less than 67.5%, with a strong probability that it is as small as 62 per cent.

SUBSIDIARY CONCLUSION (*d*):

RELATIONS BETWEEN EVAPORATION DEPTH AND MEAN TEMPERATURE.

In studying the relations between evaporation and mean temperature, it will be necessary to consider in some detail the observed evaporations at each of the evaporation stations used in Texas and New Mexico; and, while doing this, occasion will be taken to discuss the accuracy, etc., of all such evaporation measurements, and to correct all the observed evaporations (from pans of various sizes) to the corresponding evaporations from a 3-ft. square floating-pan or a 6-ft. square land-pan.

The stations on the Great Plateau at which measurements of pan evaporation had been made are Austin and El Paso in Texas; and Albuquerque, Carlsbad, Elephant Butte, and Lake Avalon, in New Mexico. There also are available for temperature evaporation studies, Piche evaporimeter records at eleven stations.

TABLE 17.—MEASURED MONTHLY EVAPORATIONS AND CORRESPONDING MEAN MONTHLY TEMPERATURES+ FOR STATIONS IN TEXAS AND NEW MEXICO.

Station	Abilene, Tex.		Austin, Tex.		Brownsville, Tex.		Corpus Christi, Tex.		El Paso, Tex.	
Elevation, in feet.	1 740 ±		500 ±		53 ±		20 ±		3 762	
Character	Floating pan *		Floating pan, 30 in		Floating pan *		Floating pan *		Floating-pan *	
Period observed	July, 1887–June, 1888		January–December, 1911.		July, 1887–June, 1888.		July, 1887–June, 1888.		July, 1887–June, 1888.	
Months.	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).
January	1.8	38.8	2.52	56.5	1.8	56.0	1.4	48.3	4.0	44.6
February	1.7	48.0	1.72	58.8	2.5	65.8	1.6	60.3	5.9	50.1
March	3.1	49.8	2.62	61.0	3.0	66.4	3.3	61.9	8.0	54.6
April	4.3	66.4	3.07	67.0	3.0	74.8	3.0	72.6	8.4	59.9
May	6.0	69.6	4.70	73.3	3.5	78.0	3.2	76.0	10.7	71.4
June	6.8	77.8	7.06	84.2	3.9	80.7	3.8	80.7	10.0	83.0
July	9.8	79.9	7.33	84.9	4.0	82.9	4.4	82.9	9.4	80.0
August	7.5	81.3	8.76	85.8	4.1	85.0	4.3	85.0	7.7	79.9
September	6.3	75.9	6.60	84.2	3.3	77.0	4.3	79.7	5.6	74.2
October	4.5	61.7	2.86	67.6	3.0	70.0	4.1	69.4	5.3	63.4
November	3.4	48.0	1.51	55.0	2.0	59.8	3.0	64.0	4.0	54.9
December	1.7	40.3	1.86	49.0	2.3	58.5	2.3	65.4	2.9	40.9
Total or mean	54.4	68.1	50.92	66.2	37.0	71.0	38.8	68.4	82.0	63.0
References	Turneaure and Russell, p. 60.	U. S. W. B.	W. S. & I. No. 280, p. 17.	U. S. W. B.	T. & R. p. 59.	U. S. W. B.	T. & R. p. 58.	U. S. W. B.	T. & R. p. 60.	U. S. W. B.
Notes.	* Based on Piche evaporimeter calculations, said to be equivalent to floating pan records.		Careful daily record on 30-in pan floating in cistern.		* See re Abilene.		* See re Abilene.		* See re Abilene.	

Station	El Paso, Tex. (at Ft. Bliss.)		El Paso, Tex. (at Ft. Bliss.)		Fort Davis, Tex.		Galveston, Tex.		Palestine, Tex.	
Elevation, in feet.	3 700 ±		3 700 ±		4 980 ±		5 ±		500 ±	
Character	Floating pan, 3 ft.		Floating-pan, 3 ft.		Floating pan.*		Floating pan *		Floating pan *	
Period observed	June, 1889–May, 1890		Sept., 1889–August, 1890.		July, 1887–June, 1888.		July, 1887–June, 1888.		July, 1887–June, 1888.	
Months.	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).
January	⊙ 3.0	45.3	⊙ 3.0	48.8	5.4	45.0	1.6	49.8	2.1	48.1
February	3.0	51.8	3.0	51.8	5.7	48.2	2.8	56.8	5.0	52.3
March	7.0	58.8	7.0	58.8	8.7	50.1	3.2	56.1	3.3	58.3
April	7.3	61.8	7.3	64.3	8.5	61.5	2.9	71.3	4.3	68.5
May	10.8	75.9	10.8	75.2	11.0	66.8	4.3	74.8	4.8	70.4
June	10.7	80.8	11.7	79.8	12.0	70.1	4.3	80.2	4.5	77.7
July	9.8	88.0	9.8	82.3	11.4	75.4	5.3	81.9	4.5	82.0
August	11.4	82.7	7.8	78.8	9.0	74.6	5.2	82.2	4.6	80.0
September	9.3	71.4	9.2	71.4	5.0	58.3	5.2	79.4	4.6	75.2
October	5.8	56.0	6.8	56.0	6.2	57.7	4.7	70.0	4.4	68.0
November	4.8	49.0	4.6	49.0	5.7	52.8	4.2	68.0	4.0	50.6
December	3.0	53.2	2.9	68.2	4.9	40.0	2.4	52.6	2.1	45.6
Total or mean	64.8	56.5	81.5	64.8	91.4	59.7	46.0	66.7	47.1	64.1
References	12th Annual Report, U. S. G. S. Pt. 2, p. 235.	U. S. W. B.	12th Annual Report, U. S. G. S. Pt. 2, p. 235.	U. S. W. B.	T. & R. p. 60.	U. S. W. B.	T. & R. p. 58.	U. S. W. B.	T. & R. p. 58.	U. S. W. B.
Notes.	The following additional record later discovered in 14th Annual Report, U. S. G. S., Pt. 2, p. 164: January, 1892 ..... 3.4 in ..... 48° February, " ..... 3.3 " ..... 50° March, " ..... 5.0 " ..... 55° April, " ..... 7.5 " ..... 64° May, " ..... 10.0 " ..... 71° June, " ..... 13.0 " ..... 81° July, " ..... 12.5 " ..... 83° August, " ..... 11.9 " ..... 79° January, 1893 ..... 4.3 " ..... 45° February, " ..... 4.8 " ..... 50° March, " ..... 4.0 " ..... 56° April, " ..... 9.5 " ..... 56°				* See re Abilene.		* See re Abilene.		* See re Abilene.	

Station	Rio Grande City, Tex. †		San Antonio, Tex.		Albuquerque, N. Mex.		Albuquerque, N. Mex.		Carlsbad, N. Mex.	
Elevation, in feet.	280 ±		700 ±		5 000 ±		5 000 ±		3 000 ±	
Character	Floating-pan.*		Floating-pan.*		Land-pan, 2 ft.		Land pan, 2 ft.		Land pan, 3 ft.	
Period observed	July, 1887–June, 1888.		July, 1887–June, 1888.		Feb., 1900–Jan., 1901.		Jan.–Dec.,* 1901.		Jan.–Dec., 1899.	
Months.	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).
January	3.7	64.7	2.4	48.0	3.04	35.4	1.81	36.3	1.20*	42.2‡
February	3.5	63.0	3.3	56.5	2.63	40.0	2.07	34.6	1.26*	42.8‡
March	3.5	64.2	4.1	56.6	5.17	49.1	5.21	47.2	3.14*	58.4‡
April	3.8	74.5	5.8	60.3	6.82	52.0	10.05	56.5	3.79*	68.8‡
May	4.5	78.0	4.0	72.8	10.08	67.0	10.96	60.4	5.58§	72.0‡
June	4.6	81.9	4.5	78.0	12.63	78.3	11.33	68.4	9.01§	78.2‡
July	6.9	81.5	[illegible]	83.6	11.78	79.1	12.36	76.2	8.09	80.6‡
August	7.0	85.4	5.8	83.0	10.21	75.0	11.73	74.3	7.90	83.6‡
September	5.3	80.2	5.2	71.5	8.00	67.7	9.65	64.0	5.60	74.6‡
October	4.9	70.9	5.4	66.9	4.58	53.8	6.62	53.9	4.20	63.6‡
November	3.8	66.1	1.2	53.5	1.73	44.7	4.21	42.4	3.22*	53.4‡
December	3.1	54.2	3.1	49.4	1.40	34.6	1.88	38.7	1.80*	48.2‡
Total or mean	53.1	71.4	52.4	66.5	77.87	56.5	87.91	54.1	54.37	63.1
References	T. & R. p. 59.	U. S. W. B.†	T. & R. p. 58.	U. S. W. B.	W. S. & I. No. 188, p. 31.	U. S. W. B.	W. S. & I. No. 188, p. 31.	U. S. W. B.	O. E. S. Bul. No. 86, p. 109.	‡U. S. W. B. §O. E. S., Bul. No. 86, p. 109. ¶Long period mean, U. S. W. B.
Notes.	* See re Abilene. † Ft. Ringgold.		* See re Abilene.		Experiments at Univ. N. Mex. published in Bull. No. 10, Vol. 3, Hadley Climatological Lab., '05. * This record continued as follows: January, 1902 ..... 2.38 in. 35.1° February, " ..... 3.92 " 39.4° March, " ..... 7.48 " 57.2° April, " ..... 10.71 " 58.9° May, " ..... 12.58 " 65.0° June, " ..... 15.08 " 71.3°				*Deduced from monthly percentage distribution relations. §These records are believed to be correct. Original text gives greater values, probably due to a confusion of cumulative evaporation with weekly depths.	

Station	Carlsbad, N. Mex.		Elephant Butte, N. Mex.		Ft. Stanton, N. Mex.		Lake Avalon, Carlsbad, N. Mex.		Santa Fé, N. Mex.	
Elevation, in feet.	3 000 ±		4 250 ±		6 150 ±		3 200 ±		7 013	
Character	Land-pan, 3 ft.		Land-pan, 4 ft.		Floating pan.*		Floating-pan, 4 ft.		Floating-pan.*	
Period observed	Jan.–Dec., 1901.		July, 1909–June, 1910.		July, 1887–June, 1888.		July, 1909–June, 1910.		July, 1887–June, 1888.	
Months.	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).
January	1.04*	44.7‡	3.5*	42.5	5.9	38.9	4.50*	46.4	3.0	30.3
February	1.04*	47.0‡	3.75*	44.3	5.9	38.0	4.50*	47.0	3.4	35.5
March	2.57*	58.9‡	4.50*	57.2	5.9	38.0	5.61	51.5	4.2	37.4
April	1.75	61.1‡	8.00*	61.8	7.8	56.8	7.45	64.4	5.8	49.9
May	5.00	71.9‡	11.50*	71.8	9.5	56.6	10.13	72.5	5.8	58.4
June	6.75	81.4‡	13.45	78.8	10.9	67.8	11.06	81.9	12.9	67.1
July	6.59	80.2‡	11.67	81.1	9.4	63.6	12.84	86.0	9.3	68.9
August	7.25	81.0‡	10.48	79.8	11.5	68.6	15.00*	79.0	9.8	65.8
September	4.75	74.1‡	8.58	70.7	6.9	60.1	9.50*	71.8	9.6	50.8
October	5.75	60.0‡	9.70	61.0	4.0	48.5	7.00*	63.3	8.7	50.0
November	1.72*	56.6‡	3.46	51.8	3.6	41.0	5.75*	37.2	6.7	41.9
December	1.14*	46.8‡	3.00*	44.4	3.6	30.0	4.60*	36.6	3.7	26.8
Total or mean	48.86	64.1	86.90	61.1	77.0	56.0	84.51	62.8	70.8	48.9
References	O. E. S. Bul. No. 119, p. 42.	‡U. S. W. B. §O. E. S., Bul. No. 119, p. 42. ¶Long period mean, U. S. W. B.	Eng. News, v. 63, p. 695.	U. S. W. B.	T. & R. p. 60.	U. S. W. B.	Eng. News, v. 63, p. 695.	U. S. W. B.	T. & R. p. 60.	U. S. W. B.
Notes.	*Deduced from monthly percentage distribution relations.		Rio Grande Dam, 10 miles west of Engle, N. Mex. *Deduced		* See re Abilene.		*Deduced.		*See re Abilene.	

⊙ Re El Paso (Ft. Bliss). All monthly depths of evaporation are measured for more than 20 days, but no month was completely measured. The depths given are the average daily for the period measured, multiplied by the number of days in the month. This applies also to records of 1892 and 1893, except that January and August, 1892, and March, 1893, were measured for less than 10 days, and April, 1893, for less than 20 days.

+ The mean monthly temperatures here used are the means of the daily mean temperatures, which are, in turn, means between the maximum and minimum each day.

A study of these data (all of which are given in Table 17, Plate XXXI) develops the fact that for each station there exists a stable average relation between its monthly depth of evaporation and its monthly mean temperature; and that on the Great Plateau the extreme relations between evaporation and temperature depart so little from the average that (where the mean temperatures are known at two places and the evaporation at only one of them) safe use may be made of this average relation to estimate the evaporation at the other.

It is well known that many other conditions besides temperature affect the depth of evaporation, such as prevalence and velocity of the wind, humidity, vapor tension, barometric conditions, etc. However, these other conditions are almost entirely unknown in most cases where engineers are called on to estimate the probable annual evaporation—whereas mean temperatures frequently are of record. It is fortunate, therefore, that the joint effect of all the climatic conditions except mean temperature is so small (either because of the relative unimportance of each, or because some tend to counteract others) that a direct comparison of mean temperature with evaporation depth at any station on the Great Plateau leads to an average relation so stable and so essentially accurate that its use is justifiable in estimating evaporation depths, even when unaided by a knowledge of the other climatic conditions which affect evaporation.

That these relations between mean monthly temperature and evaporation are stable and consistent is shown by the diagrams which have been prepared for each record station and which are given in the following discussion.

Austin, Tex.—The record of pan evaporation at Austin appears to be an excellent one. The evaporation record for an entire year of daily observation has been published,* the evaporation being that from a floating-pan 30 in. square.

From these data and from temperatures given in the records of the U. S. Weather Bureau (all as summarized in Table 17, Plate XXXI) is drawn the heavy line of Fig. 6, which shows the average relations between the monthly mean temperatures and the inches of depth of measured monthly evaporations at Austin for a floating-pan 30 in. square. The light line of Fig. 6 (derived from the heavy line by means of the relations shown by Table 2 and Fig. 5) gives those relations for a 3-ft. square floating-pan.

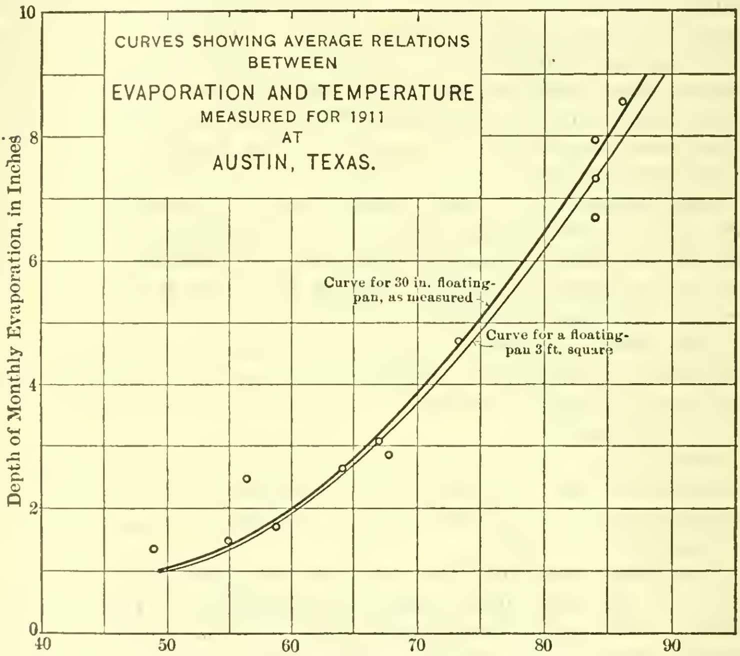
The regularity and consistency of the curves of Fig. 6 leave no doubt (at least for Austin, Tex.) of the existence of a regular relation or law between evaporation depths and mean temperatures.

El Paso, Tex.—The record at El Paso comprises evaporations as measured in 3-ft. square floating-pans for 34 months in 1889-90 to 1892-93; and 12 months Piche evaporimeter measurements in 1887-88.

* *Water Supply and Irrigation Bulletin No. 308*, p. 17.

The sources of all the El Paso data are stated in connection with their summary in Table 17 (Plate XXXI).

The record at El Paso is of much importance, as it is the nearest to Lake Conchos of any of the measurements of evaporation made in Texas and New Mexico. It happens, however, that there were many breaks in the observations, from causes not stated, that no single month's record is complete, and that the full monthly evaporations given in Table 17 (Plate XXXI) necessarily were obtained only by multiplying the average daily evaporation for the days observed each month by the number of days in the full month.



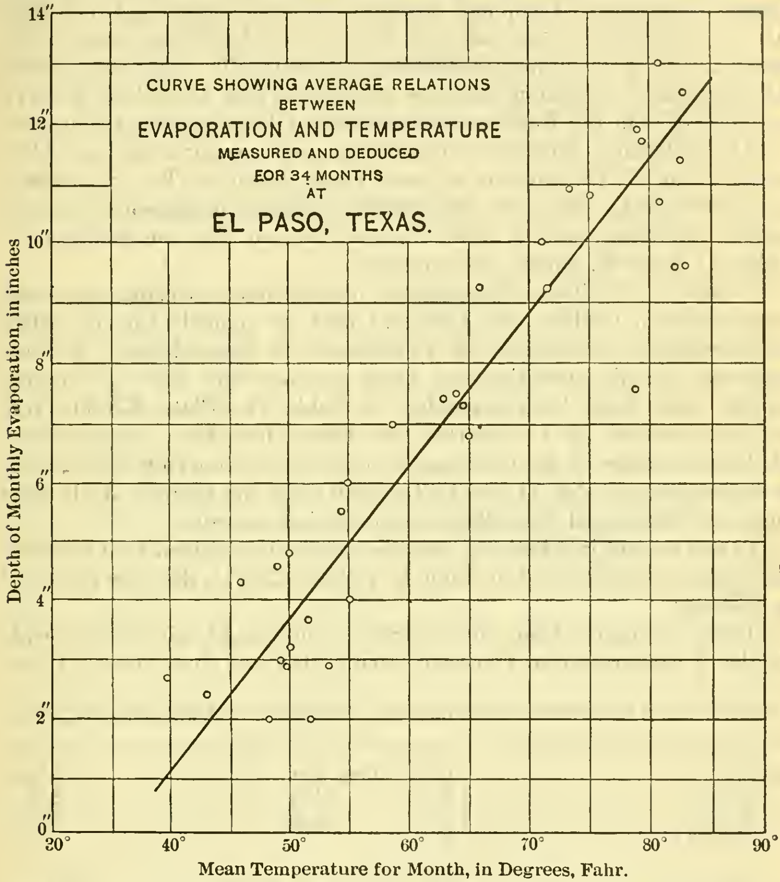
NOTE:- This record of evaporation was measured in a 30 in. floating-pan for the year 1911. The pan was at Elevation 600 ft. at Austin, Tex. For details of record see Table 17. The total measured evaporation for the year 1911, as plotted above, was (for 30-in. pan) 50.92 in., and by the Average Curve shown above (for 30 in. pan) is 51.4 in.

FIG. 6.

The relations between mean monthly temperatures and depth of monthly evaporation at El Paso are shown by the curve (in this case a straight line) of Fig. 7, which is plotted from the El Paso data of Table 17 (Plate XXXI). As for Austin (and for all the other stations studied), the curve indicates clearly a regular relation or law between temperature and depth of evaporation.

Albuquerque, N. Mex.—The published record of the evaporation at Albuquerque was found first in the Water Supply and Irrigation

Papers,* but very few data were there given as to how the observations were made. However, after much search and correspondence, there was obtained from the University of New Mexico a copy of one of the



NOTE: This record of evaporation was measured in a floating-pan, 3 ft. square, for the following periods,- May, '89 to Aug. '90; Nov. 90 to Apr., 91; Jan. to Aug., 92; and Jan. to Apr., '93; The pan was near Fort Bliss, at El Paso, Texas, at Elevation 3700 ft. ± . The measurements were in no case continuous for a full month, but in general covered more than 20 days. For details of the record see Table 17, The total depth of evaporation during the 34 months of record was 230.10 in. and the corresponding total the Average Curve shown above is 240.26 in.

FIG. 7.

Bulletins of the Hadley Climatological Laboratory of that University,† from which the facts concerning the Albuquerque evaporation observations were ascertained to be as follows:

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* *Bulletin No. 188,* p. 31.

† No. 10, Vol. III, 1905.

The observations were taken from February, 1900, to and including January, 1901, instead of for the calendar year 1900, as given in the Water Supply and Irrigation Bulletin. The second set of observations began in January, 1903, and extended to and included June, 1904. All the observations were taken in 2-ft. square land-pans, sunk in the ground, and with proper deductions for rainfall. The summarized Albuquerque evaporation data are given in Table 17 (Plate XXXI), as checked from the *Bulletin* of the Hadley Climatological Laboratory.

The relations between temperatures and evaporations at Albuquerque for all the months of record are shown on Fig. 8, where a heavy (straight) line shows the average relations as measured in 2-ft. square land-pans, and a light line as deduced (by the relations of Table 2) for 3-ft. square floating-pans.

Carlsbad, N. Mex.—Evaporation observations covering the summer periods at Carlsbad for 1899 and 1901 are published by the Office of Experiment Stations, U. S. Department of Agriculture.* For the purposes of this investigation, these evaporations for the summer months only have been expanded in Table 17 (Plate XXXI) into full-year records of 12 months, this being done by a consideration of the percentage of the total year's evaporation occurring each month, as determined on Fig. 11 and in Table 20 from the average of all other measured Texas and New Mexico evaporation records.

In one respect it is believed that the evaporations given for Carlsbad† are in error, and, as used in Table 17 (Plate XXXI), they are corrected as follows:

Table 18, taken from the record as published,‡ gives the weekly depths of evaporation at Carlsbad during May and June, 1899. Hence

TABLE 18.—Published Evaporation Records at Carlsbad, N. Mex.

May 1– 7	1.13 in.	June 4–10	2.50 in.
" 8–14	2.50 "	" 11–17	2.85 "
" 15–21	4.25 "	" 18–24	2.00 "
" 22–28	5.25 "	" 25–July 1	1.63 "
" 29–June 3	5.88 "		
Total	19.01 in.	Total	8.98 in.
Say, less 5.88 × 3/6 = 2.94 in. for June 1–3	−2.94 "	Say, plus 2.94 in. less 1.63 × 1/7 = 0.23	+2.71 "
For May, say	16.07 in.	For June, say	11.69 in.

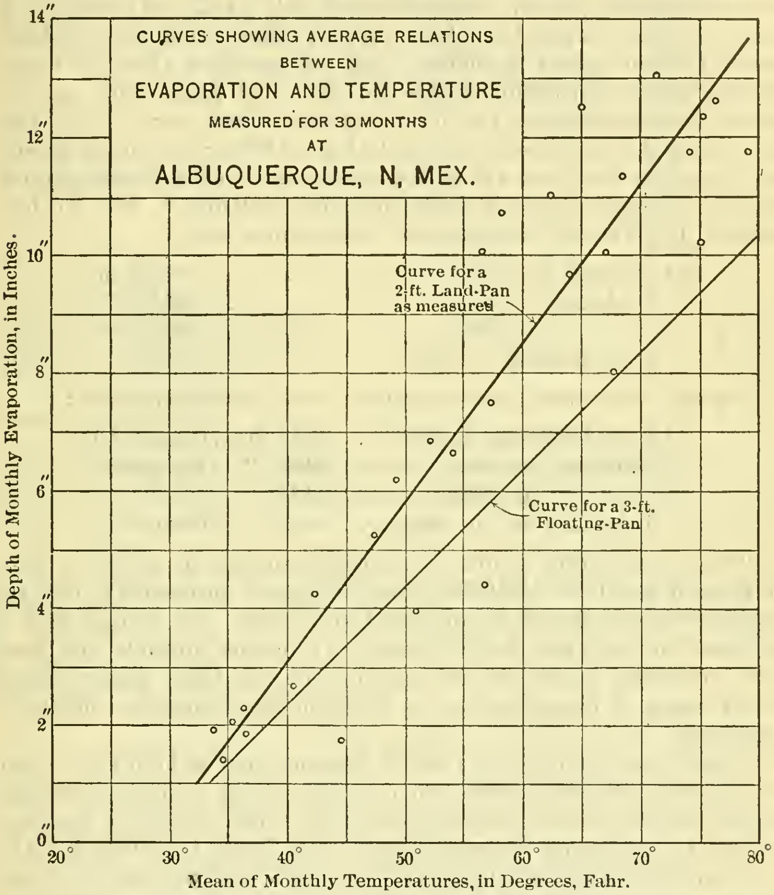
the published records indicate that the evaporation at Carlsbad was about 16 in. for May, and nearly 12 in. for June, 1899.

* *Bulletins Nos. 86* and *119*.

† *Bulletin No. 86,* p. 109.

‡ In *Bulletin No. 86*.

Such great monthly evaporations seem to be very improbable, as Carlsbad is not a station having a record (except for the 2 months, May and June, 1899) which shows excessive monthly evaporation depths, and as the depths for May and June, 1901, were only 5.00 and



NOTE:-This record of evaporation was measured in a 2-ft. Land-Pan, during the periods from Feb.,1900,to Jan., 1901, incl., and Jan., 1903, to June, 1904, incl. For details of the record see Table 17.

The total evaporation during the 30 months of measurement was 215.72 in. (for 2-ft. pan) and for the same months by the Average Curve above (for 2-ft. pan) 218.54 in.

Fig. 8.

6.75 in., respectively, and not materially different from the depths for July and August. Also, the evaporation depths for May and June, 1901, at Carlsbad appear to be reasonable because of the similar depths at Lake McMillan, not far distant.

Another cause for suspicion of the correctness of the published record at Carlsbad is the much greater evaporation depth observed at Lake Avalon, Carlsbad (see Table 17, Plate XXXI). The Lake Avalon record gives 94.51 in. for 12 months' evaporation in 1909-10, as compared with yearly evaporations of only 54.37 and 43.26 in. at Carlsbad. This near-by Lake Avalon record (almost double the Carlsbad records) might appear to indicate that the published Carlsbad record showing great evaporation depths for May and June, 1899, may be correct, notwithstanding the inconsistencies noted, were it not that the May and June depths at Carlsbad for 1899 are so much greater than those for 1901, and are so disproportionate to the depths of other near-by stations, such as Roswell and Lake McMillan.* For the four months, July-October, the recorded evaporations were:

At Roswell, in 1901	15.55 in.
" Carlsbad, in 1901	22.25 "
" " in 1899	25.72 "
" Lake Avalon, in 1909	41.38 "

For the five months, May-September, the evaporations were:

At Lake McMillan, in 1901	27.84 in.	(Recorded.)
" Carlsbad, in 1901	30.25 "	(Adopted.)
" " in 1899	36.11 "	(")
" Lake Avalon, in 1909	55.55 "	(Recorded.)

These evaporation depths at Carlsbad, already in excess of those at Roswell and Lake McMillan, make it appear unreasonable that the Carlsbad depths should be increased still more, even though this is indicated by the Lake Avalon record. It appears probable that there was some local peculiarity of exposure in the Lake Avalon record which makes it inapplicable as a check on the evaporation depths at Carlsbad.

Finally, no record in the Plateau Region shows as high an evaporation as 16 in. for any month, nor does any record even approach such a great depth for May, the evaporation in which usually is less than that of the following summer months (see Table 17, Plate XXXI).

From the foregoing discussion, it appears probable that the published evaporation record of Carlsbad, N. Mex., for May and June, 1899, is in error, and gives depths which are excessive.

In an attempt to correct the record, a study of the weekly evaporations for May and June, 1899, as published, makes it appear probable that the error was in using the weekly evaporations between May 7th and June 3d cumulatively instead of separately, probably by a clerical error.† On this assumption, the published records are corrected as in Table 19.

* *Bulletin No. 119,* Office of Experiment Stations, p. 42.

† See note under Table 19.

TABLE 19.—CORRECTED RECORDS FOR CARLSBAD, N. MEX.

Dates.	Published.*	Corrected.
May 1- 7	1.13 in.	1.13 in.
" 8-14	2.50 "	1.37 "
" 15-21	4.25 "	1.75 "
" 22-28	5.25 "	1.00 "
" 29-June 3	5.88 "	0.63 "
Totals	19.01 in.	5.88 in.
June 4-10	2.50 in.	2.50 in.
" 11-17	2.85 "	2.85 "
" 18-24	2.00 "	2.00 "
" 25-July 1	1.63 "	1.63 "
Totals	8.98 in.	8.98 in.

May 1-June 3 (corrected)	5.88 in.
less June 1-3 = 0.63 × 3/6 (?), say	−0.31
Corrected evaporation for May, 1899	5.57 in.
June 4-July 1 (published and corrected)	8.98 in.
Plus June 1-3, 0.30 in., and less July 1, 1.63 × 1/7 = 0.23 in.	0.07 in.
Corrected for June, 1899	9.05 in.

* It is noticeable that the weekly evaporation increases rapidly and continuously up to June 3d, and then drops suddenly. (By some error in the computation, the values obtained and used in the investigation were 5.58 in. for May and 9.01 in. for June.)

The average relations between temperatures and evaporations at Carlsbad are shown by the diagram, Fig. 9, where the months actually observed are shown by heavy circles, those only deduced by light circles, the observed relations for 3-ft. square land-pans by a heavy curve, and the deduced relations for 3-ft. square floating-pans (assumed as 80% of the land-pans) by a light curve.

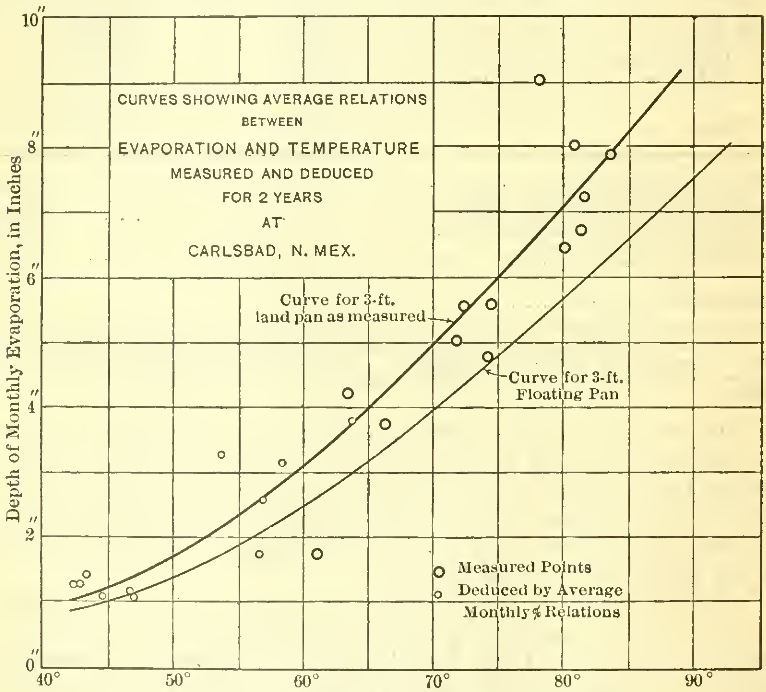
Elephant Butte, N. Mex.—The record of evaporation at this station is taken from the summary of Professor Bigelow's investigation.* As shown in Table 17 (Plate XXXI), the evaporations were measured for the 6 months, June-November, 1909, in a 4-ft. square land-pan. The evaporations of the 6 months, December-May, were deduced by some method not well described. The deductions appear to have been made properly, however, from the fact that the deduced values for the missing months show the same percentage relations to the entire yearly evaporation as those shown on Fig. 11 and in Table 20 for the average of all the measured evaporation stations.

The temperature evaporation relations for Elephant Butte are shown by diagram on Fig. 10, where the observed and deduced values are distinguished, and the relations are shown for both a 4-ft. square land-pan and a 3-ft. square floating-pan.

* As given in *Engineering News*, Vol. 63, pp. 694–695.

Because of its comparative nearness, that at Elephant Butte is perhaps the most important (next to that at El Paso) of any of the Texas and New Mexico evaporation records available for the determination of the probable evaporation at Lake Conchos.

Lake Avalon, N. Mex.—This evaporation record, also, is taken from Professor Bigelow's investigation.* As shown there and in Table 17 (Plate XXXI), the Lake Avalon evaporations were measured only for the 5 months, March-July, 1909, while those for the 7 months, August-February, were deduced by some method not clearly stated.



NOTE;- This record of evaporation was measured in a 3-ft. land-pan during 1899 and 1901. The elevation of Carlsbad is 3000 ± ft.
The details of this record, part of which is deduced, are given on Table 17.

The total evaporation during the 13 months of actual measurement was ______(for land-pan)____76.06 in. and for the same months by the Average Curve shown above (for land-pan) was 77.51 in.

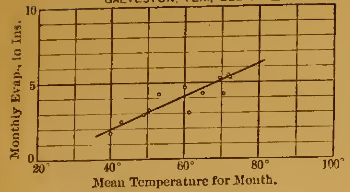
FIG. 9.

As pointed out in the discussion of the Carlsbad records, there is a disagreement between the Lake Avalon records and those for land-pans at Carlsbad. The latter are checked both for evaporation depth and for percentage distribution among the months by the independent records at the near-by stations of Roswell and Lake McMillan; and as

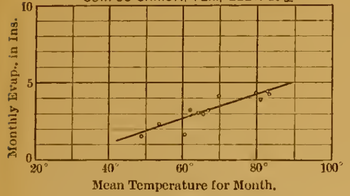
* *Engineering News*, Vol. 63, pp. 694–695.

CURVES SHOWING RELATIONS BETWEEN
EVAPORATION AND TEMPERATURE
AS DETERMINED BY
PICHE EVAPORIMETER RECORDS
IN
TEXAS AND NEW MEXICO
1887-1888.

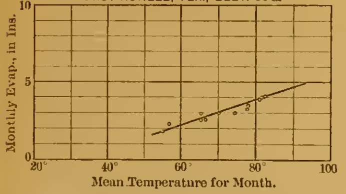
GALVESTON, TEX., ELEV. 6'±



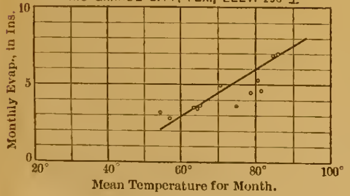
CORPUS CHRISTI, TEX., ELEV. 20'±



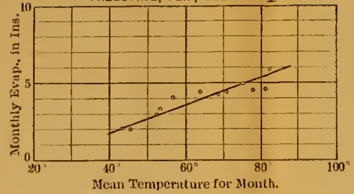
BROWNSVILLE, TEX., ELEV. 50'±



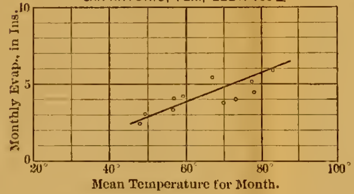
RIO GRANDE CITY, TEX., ELEV. 230'±



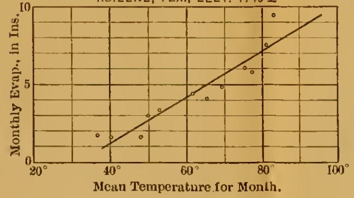
PALESTINE, TEX., ELEV. 500'±



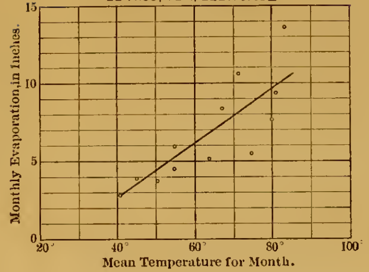
SAN ANTONIO, TEX., ELEV. 700'±



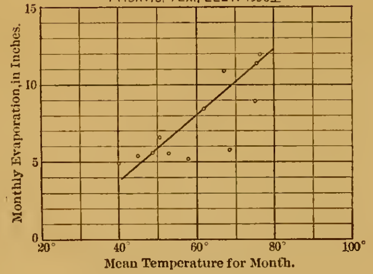
ABILENE, TEX., ELEV. 1740'±



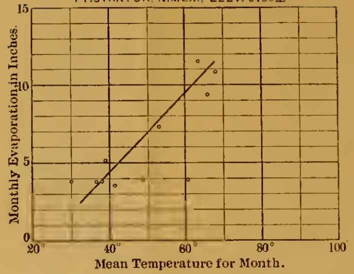
EL PASO, TEX., ELEV. 3762±



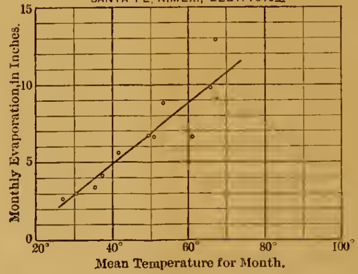
FT. DAVIS, TEX., ELEV. 4930±



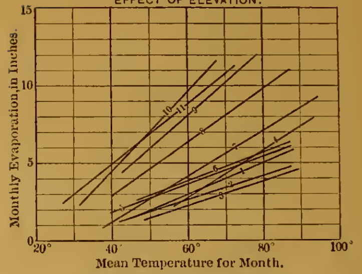
FT. STANTON, N. MEX., ELEV. 6150±



SANTA FE, N. MEX., ELEV. 7013±



COMBINED
TEMPERATURE-EVAPORATION CURVES
SHOWING
EFFECT OF ELEVATION.



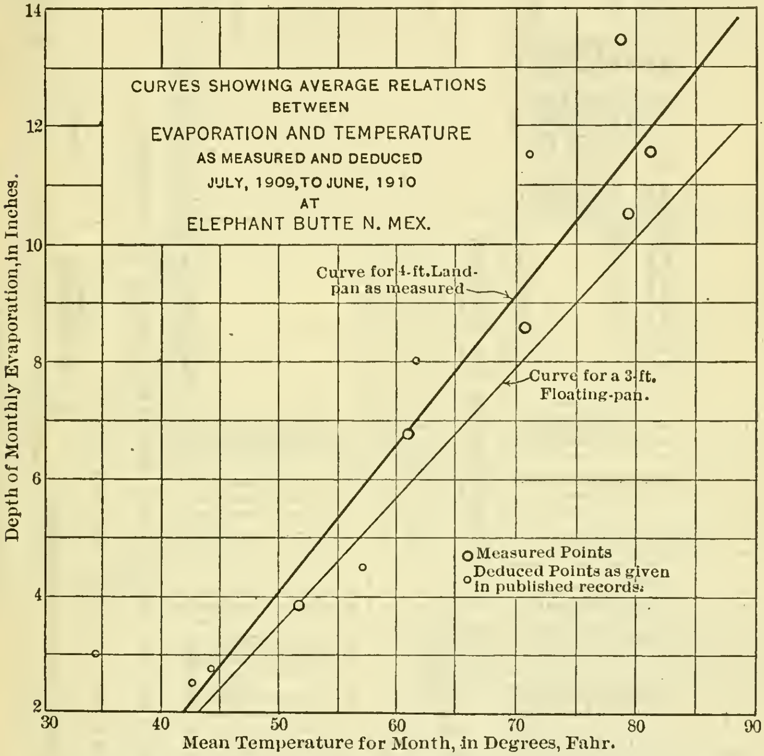
The above curves are numbered in the order of elevation above Sea Level as follows:
Curve 1, elev. 6; Curve 5, elev. 500; Curve 9, elev. 4930;
Curve 2, " 20; Curve 6, " 700; Curve 10, " 6150;
Curve 3, " 50; Curve 7, " 1740; Curve 11, " 7013;
Curve 4, " 230; Curve 8, " 3762;

Notes.-All Temperatures are in Degrees, Fahrenheit.

The records of evaporation plotted hereon are from calculations by the United States Signal Service, based on Piche Evaporimeter records for 1887-88. For the detailed records see Table 17. The depths of evaporation here given are said to be equivalent to "water surface" evaporation, and probably correspond fairly to floating-pan records. Subsequent measurements in pans, at scattered stations in the United States, are said to indicate that the Piche records give somewhat greater depths of evaporation than those measured.

Lake Avalon is near Carlsbad, an attempt is made here to analyze the disagreement between their records.

As shown in Table 17 (Plate XXXI), the total year's evaporation at Lake Avalon was 94.51 in., as published. If the evaporation depths for the 7 missing months are deduced by use of the mean monthly percentage distributions for all measured evaporation stations on the



NOTES:- This record of evaporation was measured in a 4-ft. Land-pan for the season July, 1909, to June, 1910, incl. The pan was at Elephant Butte, on the Rio Grande River, near Engle, N.Mex., at Elevation 4250 ft.±. The details of the record are given on Table 17.

A portion of the season's record was deduced, but the published record does not state the manner in which this was done.

The total evaporation during the months of actual measurement was (for 4-ft. pan) 54.7 in. and for the same months by the Average Curve shown above (for 4-ft. pan) 55.6 in.

Fig. 10.

Plateau (see Fig. 11 and Table 20), the year's evaporation depth would be reduced to 85.8 in. In Fig. 12, the evaporation temperature curve for Lake Avalon (a straight line) is drawn by considering the points of actual measurement only, neglecting the seven deduced points; and if the depths for the missing 7 months are supplied from that diagram, the total year's evaporation will be approximately 87 in.

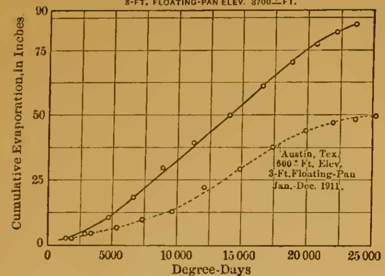
TABLE 20.—PERCENTAGE DISTRIBUTION OF EVAPORATION BY MONTHS FOR STATIONS IN TEXAS AND NEW MEXICO.

Station.	Season.	Character of record.	Percentage of Total Annual Evaporation, by Months. Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total.
RECORDS BY PICHE EVAPORIMETER, TAKEN IN 1887–1888.															
Abilene, Tex.	July, 1887–June, 1888.	Piche Evaporimeter.	3.3	3.1	5.7	7.7	9.2	10.7	17.5	13.8	11.4	8.3	6.2	3.1	100
Brownsville, Tex.	" " – " "	" "	4.9	7.0	7.9	8.1	9.5	10.5	10.8	11.1	8.9	8.1	7.0	6.2	100
Corpus Christi, Tex.	" " – " "	" "	3.6	4.1	8.5	7.7	8.3	10.1	11.3	11.1	11.1	10.6	7.7	5.9	100
El Paso, Tex.	" " – " "	" "	4.9	4.8	7.3	10.2	13.1	16.6	11.5	9.4	6.8	6.3	5.6	3.5	100
Fort Davis, Tex.	" " – " "	" "	5.9	6.2	7.3	9.3	12.0	13.1	12.5	9.9	6.5	5.7	6.2	5.4	100
Galveston, Tex.	" " – " "	" "	3.5	6.1	7.0	6.3	9.4	9.1	11.5	11.3	11.3	10.2	9.1	5.2	100
Palestine, Tex.	" " – " "	" "	4.5	6.4	7.0	8.9	9.1	9.6	12.2	9.8	10.2	9.3	8.5	4.5	100
Rio Grande, Tex.	" " – " "	" "	5.1	6.6	6.6	6.7	8.5	8.7	13.0	13.2	9.8	9.2	6.8	5.8	100
San Antonio, Tex.	" " – " "	" "	4.6	6.3	7.8	7.2	7.6	8.6	12.7	11.1	9.9	10.3	8.0	5.9	100
Fort Stanton, N. Mex.	" " – " "	" "	5.1	5.1	6.7	9.5	12.3	14.1	12.2	15.1	5.1	5.2	4.7	4.9	100
Santa Fé, N. Mex.	" " – " "	" "	3.7	4.3	5.3	8.5	11.0	16.2	11.5	12.3	8.3	8.4	7.1	3.4	100
Average.			4.5	5.5	7.0	8.2	10.0	11.6	12.4	11.6	9.0	8.3	7.0	4.9	100
Maximum departures +			+1.4	+1.5	+1.5	+2.0	+3.1	+5.0	+5.1	+2.2	+2.4	+2.3	+2.1	+1.3	
Maximum departures –			–1.2	–2.4	–1.7	–1.5	–2.4	–3.0	–1.6	–2.2	–3.9	–3.1	–2.3	–1.8	
RECORDS BY MEASUREMENTS IN PANS.															
Austin, Tex.	Year of 1911.	Floating-pan, 30 ft.	4.9	3.4	5.2	6.0	9.2	15.6	14.4	16.9	13.1	5.6	3.0	2.7	100
El Paso, Tex. (Ft. Bliss).	May, 1889–Apr., 1890.	" " 3 ft.	2.4	2.4	8.3	8.7	12.7	12.7	11.4	13.5	10.9	8·1	5.5	3.4	100
	June, 1889–May, 1890.	" " "	2.4	2.4	8.3	8.7	12.7	12.7	11.4	13.5	10.9	8.1	5.5	3.4	100
	July, 1889–June, 1890.	" " "	2.3	2.3	8.2	8.6	12.7	13.6	11.3	13.4	10.8	8.0	5.4	3.4	100
	Aug., 1889–July, 1890.	" " "	2.3	2.3	8.2	8.6	12.7	13.6	11.3	13.4	10.8	8.0	5.4	3.4	100
	Sept., 1889–Aug., 1890	" " "	2.5	2.5	8.6	9.0	13.2	14.4	11.7	9.3	11.3	8.3	5.6	3.6	100
Albuquerque, N. Mex.	Feb., 1900–Jan., 1901.	Land-pan, 2 ft.	2.6	3.4	7.9	8.8	12.8	16.3	15.2	13.1	10.3	5.6	2.2	1.8	100
	Year of 1903.	" " " "	2.1	2.4	5.9	11.4	12.5	12.9	14.1	13.3	11.0	7.5	4.8	2.1	100
Elephant Butte, N. Mex.	July, 1909–June, 1910	Land-pan, 4 ft.	2.9	3.2	5.2	9.2	13.2	15.5	13.3	12.0	9.9	7.8	4.4	3.4	100
Lake Avalon, N. Mex.	July, 1909–June, 1910	Floating-pan, 4 ft.	4.8	4.8	5.8	7.9	10.7	11.2	13.6	12.8	10.1	7.9	6.1	5.3	100
Average.			2.9	2.9	7.2	8.7	12.2	13.8	12.8	13.1	10.9	7.5	4.8	3.2	100
Maximum departures +			+2.0	+1.9	+1.4	+2.7	+1.0	+2.5	+2.4	+3.8	+2.2	+0.8	+1.3	+2.1	
Maximum departures –			–0.8	–0.6	–2.0	–2.7	–3.0	–2.6	–1.5	–3.8	–1.0	–1.9	–2.6	–1.4	

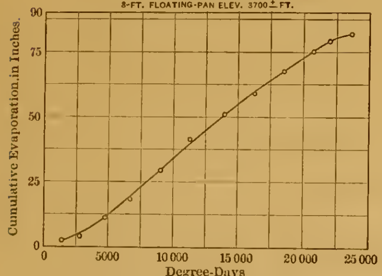
For data from which above percentages were calculated see Table 17.

DIAGRAMS SHOWING
RELATIONS BETWEEN
EVAPORATION
AND
TEMPERATURE
CONSIDERED CUMULATIVELY
AT STATIONS WHERE MEASURED
IN TEXAS AND NEW MEXICO.

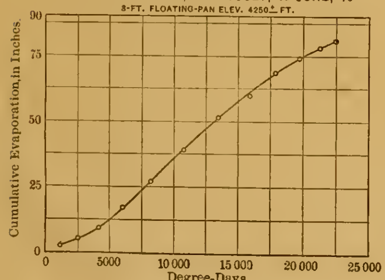
EL PASO, TEX. JUNE 1889-MAY 1890
3-FT. FLOATING-PAN ELEV. 3700± FT.



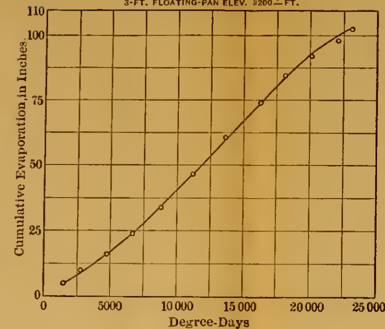
EL PASO, TEX. SEPT. 1889-AUG. 1890
3-FT. FLOATING-PAN ELEV. 3700± FT.



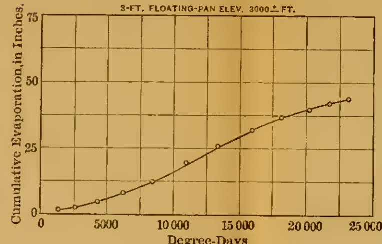
ELEPHANT BUTTE, N.MEX. JULY, '09-JUNE, '10
3-FT. FLOATING-PAN ELEV. 4250± FT.



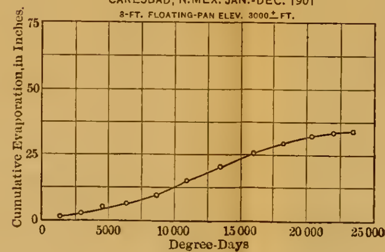
LK. AVALON, N.MEX. JULY, '09-JUNE, '10
3-FT. FLOATING-PAN ELEV. 3200± FT.



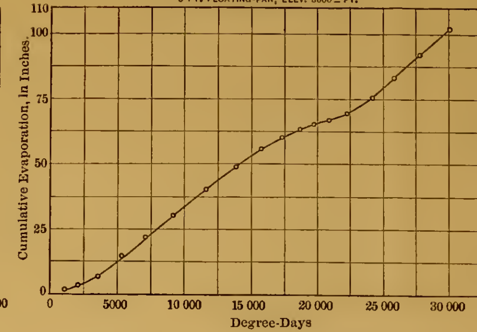
CARLSBAD, N.MEX. JAN.-DEC. 1899
3-FT. FLOATING-PAN ELEV. 3000± FT.



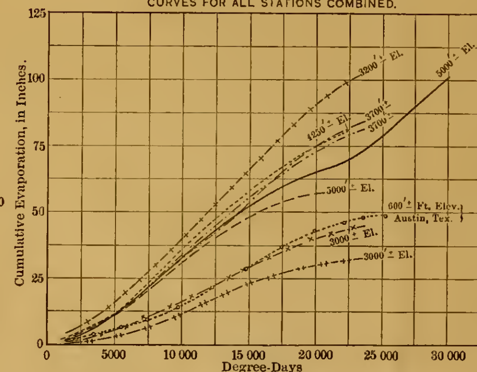
CARLSBAD, N.MEX. JAN.-DEC. 1901
3-FT. FLOATING-PAN ELEV. 3000± FT.



ALBUQUERQUE, N.MEX. JAN. 1903-JULY, 1904
3-FT. FLOATING-PAN, ELEV. 5000± FT.



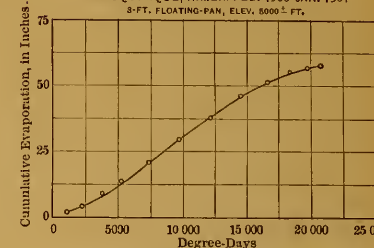
EVAPORATION-TEMPERATURE RELATIONS
CURVES FOR ALL STATIONS COMBINED.



DESCRIPTION

Lake Avalon, N.Mex., July 1909-June, '10.
Elephant Butte, July 1909-June 1910.
El Paso, Tex., June 1889-May 1890.
El Paso, Tex., Sept. 1889-Aug. 1890.
Albuquerque, New Mex., Jan. 1903-July, 1904.
Albuquerque, New Mex., Feb. 1900-Jan. 1901.
Carlsbad N.Mex., January-Dec. 1899.
Carlsbad N.Mex., January-Dec. 1901.

ALBUQUERQUE, N.MEX. FEB. 1900-JAN. 1901
3-FT. FLOATING-PAN, ELEV. 5000± FT.



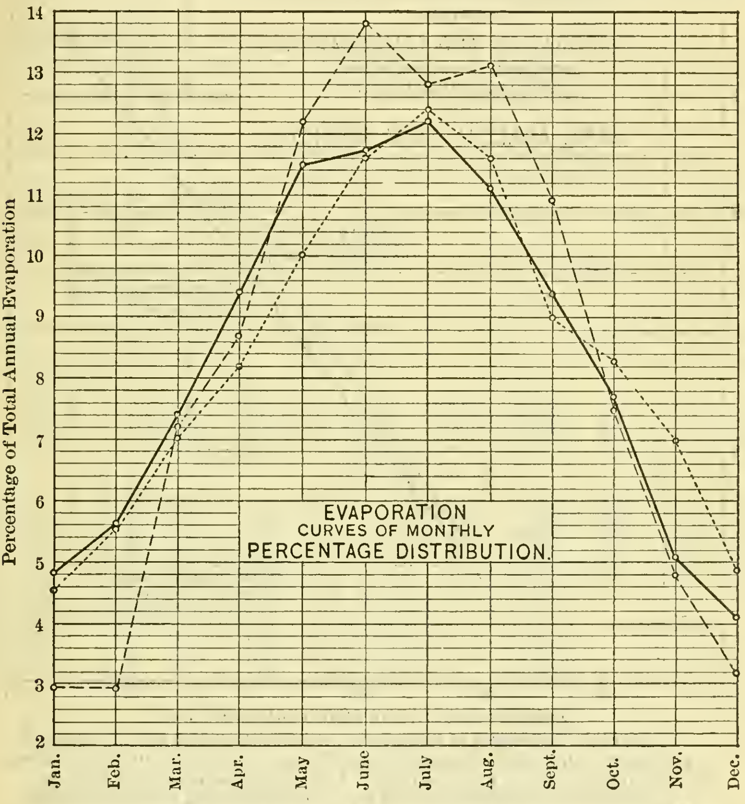
NOTES.

The data in Table 21 were taken from Table 17 in which full notes and references are given.

The measured depths of evaporation as given in that Table have been modified by the application of coefficients, in order that all records may conform to depths for a 3-ft. floating-pan, and thus be directly comparable.

All records are plotted hereon beginning with January and ending with December, of the year observed, except that of Albuquerque, N. Mex., for 1903-04 which extends to July, 1904.

However, even with the reduction of the year's evaporation at Lake Avalon to, say, 86 to 87 in., its evaporation still would be so much greater than that of Carlsbad as to leave some doubt as to the true evaporation in this vicinity. The most probable explanation of the disagreement between the Lake Avalon and the Carlsbad records seems to be that during the particular year under discussion the evaporations



-------Mean distribution from records of eleven stations in Texas and New Mexico taken with Piche evaporimeter in 1887-88.

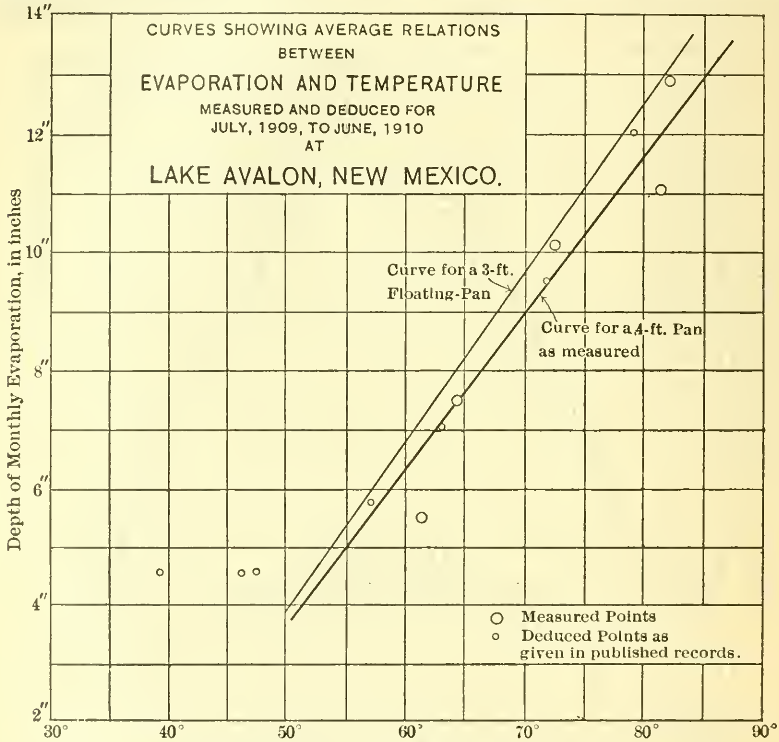
— — —Mean distribution from records of measured evaporation at five stations in Texas and New Mexico for various years.

———Distribution at La Boquilla, April, 1912,-December, 1913, as calculated by temperture-evaporation relations.

FIG. 11.

at the two places (not far apart) were abnormal, those at Lake Avalon being abnormally high and those at Carlsbad abnormally low. In considering hereinafter the effect of elevation on temperature, the means of the Carlsbad and Lake Avalon evaporations are used as the Carlsbad values, and all other considerations of the Lake Avalon records are omitted.

The curves showing the relations between the mean monthly temperatures and the monthly evaporation depths at Austin, El Paso, Albuquerque, Carlsbad, and Elephant Butte, shown separately on Figs. 6 to 10, also are shown together for comparison on Diagram I of Fig. 13.



NOTES: This record of evaporation was measured in a 4-ft. floating-pan for the season, July, 1909, to June, 1910, incl. The pan was floating in Lake Avalon, near Carlsbad, N. Mex. at Elevation 3200 ft.±. For details of record see Table 17.

A portion of the season's record as given was deduced, but the published record does not state the manner in which this was done. The total evaporation during the months of actual measurement was for (4-ft. pan) .... 47.01 in. and for the same months by the average curve shown above (for 4-ft. pan) 47.7 in.

FIG. 12.

Piche Evaporimeter Records.—In addition to the six stations for which pan evaporations are given in Table 17 (Plate XXXI), that table presents evaporimeter observations at one of the six (El Paso) and at ten other stations in Texas and New Mexico. These evaporimeter records* are stated to have been published in the *Monthly Weather Review* of the U. S. Signal Service, September, 1888, p. 235.

* "Public Water Supplies", Turneaure and Russell (1911), Second Edition, p. 60.

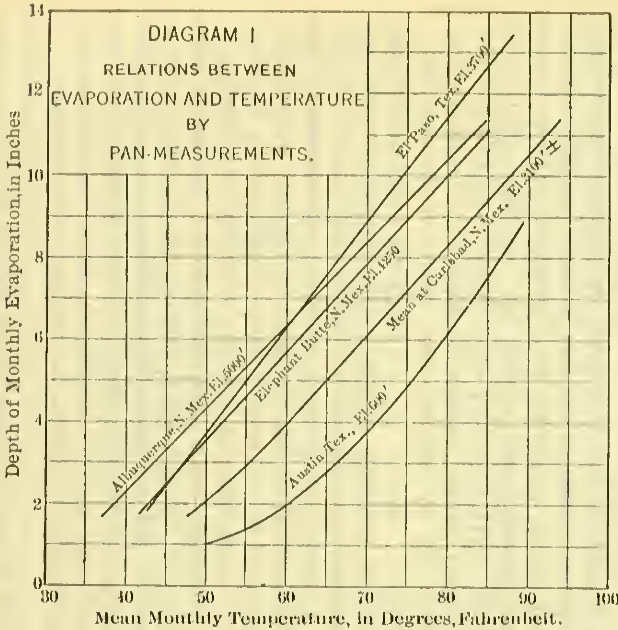
TABLE 21.—CUMULATIVE EVAPORATION AND TEMPERATURE AT STATIONS FOR WHICH EVAPORATION WAS MEASURED IN TEXAS AND NEW MEXICO.

STATION.	EL PASO, TEX. (AT FT. BLISS).				EL PASO, TEX. (AT FT. BLISS).				ELEPHANT BUTTE, N. MEX.			
Elevation	3 700 ft.±				3 700 ft.±				4 250 ft ±			
Character	Floating-pan, 3 ft.				Floating-pan, 3 ft.				Land-pan, 4 ft., modified to 3-ft., floating.*			
Period observed	June, 1889–May, 1890, incl.				Sept., 1889–Aug., 1890, incl.				July, 1909–June, 1910, incl.			
	Evaporation.		Temp. (F°).		Evaporation.		Temp. (F°).		Evaporation.		Temp. (F°).	
Months.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.
January	2.0	2.0	48.8	1 497	2.0	2.0	48.8	1 497	2.8	2.3	42.5	1 316
February	2.0	4.0	51.8	2 948	2.0	4.0	51.8	2 948	2.8	4.9	44.2	2 555
March	7.0	11.0	58.8	4 771	7.0	11.0	58.8	4 771	4.2	9.1	57.2	4 328
April	7.8	18.8	64.8	6 699	7.3	18.8	64.3	6 699	7.4	16.5	61.6	6 176
May	10.8	29.1	75.2	9 031	10.8	29.1	75.2	9 031	10.7	27.2	71.2	8 384
June	10.7	39.8	80.8	11 454	11.7	40.8	79.6	11 419	12.5	39.7	78.8	10 747
July	9.6	49.4	83.0	14 028	9.5	50.4	82.3	13 970	10.8	50.5	81.1	13 262
August	11.4	60.8	82.7	16 691	7.6	58.0	78.8	16 413	9.8	60.3	79.2	15 717
September	9.2	70.0	71.4	18 733	9.2	67.2	71.4	18 555	8.1	68.4	70.7	17 838
October	6.8	76.8	65.0	20 748	8.8	74.0	65.0	20 570	6.8	74.7	61.0	19 729
November	4.8	81.4	49.0	22 218	4.6	78.6	49.0	22 040	3.6	78.3	51.8	21 283
December	2.9	84.3	53.2	23 863	2.9	81.5	53.7	23 689	2.8	81.1	34.4	22 349
Totals	84.8	84.8	65.8	23 868	81.5	81.5	64.8	23 689	81.1	81.1	61.1	22 349

STATION.	LAKE AVALON (CARLSBAD, N. MEX.).				CARLSBAD, N. MEX.				CARLSBAD, N. MEX.			
Elevation	3 200 ft.±				3 000 ft.±				3 000 ft.±			
Character	Floating-pan, 4 ft., modified to 3 ft.†				Land-pan, 3 ft., modified to floating.‡				Land-pan, 3 ft., modified to floating.§			
Period observed	July, 1909–June, 1910, incl.				Jan.–Dec., 1899.				Jan.-Dec., 1901.			
	Evaporation.		Temp. (F°).		Evaporation.		Temp. (F°).		Evaporation.		Temp. (F°).	
Months.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.
January	4.9	4.9	46.4	1 438	1.01	1.01	42.8	1 308	0.83	0.83	44.7	1 386
February	4.9	9.8	47.6	2 771	1.01	2.02	42.8	2 507	0.83	1.66	47.0	2 702
March	5.9	15.7	61.3	4 672	2.51	4.53	68.4	4 317	2.06	3.72	56.9	4 466
April	8.0	23.7	64.4	6 608	3.03	7.56	63.8	6 231	1.40	6.12	61.1	8 299
May	10.9	34.6	72.3	8 845	4.46	12.02	72.6	8 482	4.00	9.12	71.9	8 528
June	11.9	46.5	81.2	11 281	7.21	19.23	78.2	10 828	5.40	14.52	81.4	10 969
July	13.9	60.4	82.0	13 823	6 42	25.65	80.6	13 326	5.20	19.72	80.2	13 456
August	13.0	73.4	79.0	16 272	6.32	31.97	83.6	15 918	5.80	25.62	81.6	15 985
September	10.2	83.6	71.8	18 426	4.48	36.45	74.5	18 163	3.80	29.32	74.1	18 208
October	7.5	91.1	83.2	20 385	3 36	39.81	68.6	20 124	3.00	32.32	66.6	20 273
November	6.2	97.3	57.2	22 101	2.57	42.38	53.4	21 726	1.38	33.70	56.6	21 971
December	4.9	102.2	39.6	23 329	1.11	43.49	48.2	23 066	0.91	34.61	46.8	23 422
Totals	102.2	102.2	63.8	23 329	43.49	43.49	63.1	23 066	34.61	34.61	64.1	23 422

STATION.	ALBUQUERQUE, N. MEX.				ALBUQUERQUE, N. MEX.							
Elevation	5 000 ft.±				5 000 ft.±							
Character	Land-pan, 2 ft., modified to floating, 3 ft.‖				Land-pan, 2 ft., modified to floating-pan, 3 ft.¶							
Period observed	Feb., 1900–Jan., 1901.				January 1, 1903–July 1, 1904.							
	Evaporation.		Temp. (F°).		Evaporation.		Temp. (F°).		Evaporation.		Temp. (F°).	
Months.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.
January	1.61	1.61	35.4	1 097	1.34	1.34	36.3	1 125	1.69	66.73	36.4	20 905
February	1.95	3.46	40.6	2 234	1.53	2.87	34.6	2 094	2.90	69.63	50.8	22 378
March	4.56	8.02	49.1	3 756	3.86	8.73	47.2	3 557	6.54	76.17	57.2	24 151
April	5.05	13.07	52.0	5 316	7.43	14.18	56.5	5 252	7.93	83.10	58.2	25 897
May	7.46	20.58	67.0	7 393	8.12	22.28	62.4	7 186	9.27	92.37	65.0	27 912
June	9.35	29.88	76.8	9 682	8.38	30.66	68.4	9 238	9.64	102.01	71.3	30 051
July	8.72	38.60	79.1	12 134	9.15	39.81	75.2	11 569				
August	7.66	46.18	75.0	14 459	8.68	48.49	74.2	13 869				
September	6.92	52.08	67.7	16 490	7.14	65.63	64.0	15 789				
October	3.24	55.32	56.8	18 251	4.90	60.63	53.9	17 460				
November	1.28	58.60	44.7	19 692	3.12	63.65	42.4	18 732				
December	1.04	57.64	34.8	20 665	1.39	65.04	33.7	19 777				
Totals	57.64	57.64	56.5	20 665	65.04	65.04	54.1	19 777				

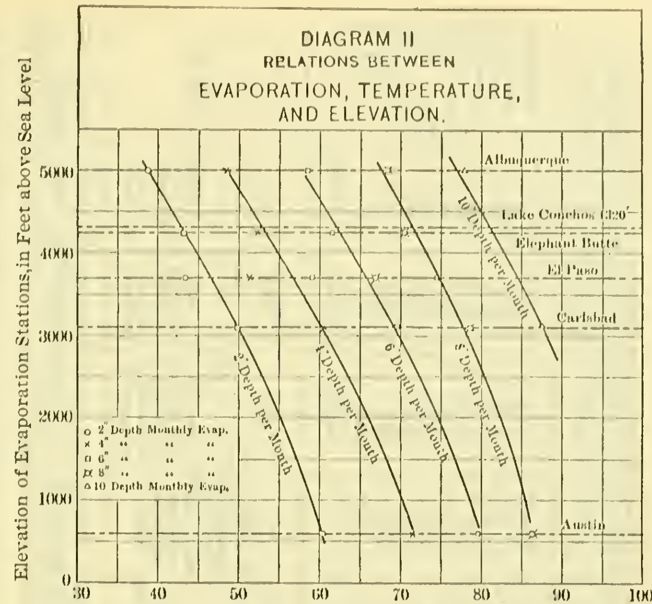
* Recorded Evap. × 1 08 × 0.80. † Recorded Evap. × 1.08. ‡ Land-pan Record × 80%. § Land-pan Record × 80%. ‖ Land-pan Record × $0.80 \times \frac{1}{1.08}$. ¶ Land-pan Record × $0.80 \times \frac{1}{1.08}$.



NOTE:- On Diagram I are plotted the curves of relation between Evaporation and Temperature for all stations in Southern Texas and New Mexico at which pan-evaporation measurements are known to have been made.

For reasons stated in the text, the record for Carlsbad, N. Mex., shown hereon is the mean of the curves resulting from records at Carlsbad in 1899 and 1901, and at Lake Avalon, near Carlsbad, in 1909-10.

Diagram I shows that, in general, the rate of evaporation at any temperature increases with elevation.



NOTE:- Diagram II is derived from Diagram I, and shows that the mean monthly temperature required to produce a given depth of monthly evaporation decreases with elevation.

The curves show a definite relation between elevation and temperature necessary to produce any given evaporation depth, to which all stations conform closely except El Paso, Tex. The record at that station though covering a period of 34 months, is not complete for any month, the missing days of each month having been deduced in every case. Although portions of the records at most other stations were also deduced, it is believed to be proper to give greatest weight to those records which conform to definite relations, and the curves of Diagram II have been so drawn.

FIG. 13.

and to have been records of observations made in the year, July, 1887-June, 1888.

The evaporation temperature relations at all these eleven evaporimeter stations are shown on Plate XXXII by eleven separate diagrams and one combined diagram. These evaporimeter diagrams show clearly (as do those of the pan-evaporations at the six stations) that the monthly depth of evaporation varies with the monthly mean temperature, in general in accordance with a "straight-line" relationship.

Cumulative Relations Between Monthly Mean Temperatures and Monthly Evaporations.—As a further aid to the study of the law of temperature evaporation relations, the records of pan evaporations at the six stations in Texas and New Mexico given in Table 17 (Plate XXXI) were worked up "cumulatively" or as mass-diagrams, and are so presented on the eight separate diagrams and the one combined diagram of Plate XXXIII.

In working up these mass-diagrams in the consideration of the actual problem, the record for Austin, Tex., in Table 17 (Plate XXXI) inadvertently was omitted from Plate XXXIII. To make the present paper more complete, the Austin mass-diagram has been added to Plate XXXIII in dotted lines, on one of the El Paso diagrams and on the combined diagram, and the tabulated cumulative relations for Austin are presented in Table 22 as an example of the working out of the cumulative method.

TABLE 22.—CUMULATIVE EVAPORATION-TEMPERATURE RELATIONS AT AUSTIN, TEX., ELEVATION 600 FT. ±, JANUARY-DECEMBER, 1911.

Month.	Observed in 30-in. floating-pan (from Table 17).	Deduced for 3-ft. square floating-pan.	Cumulative evaporation for 3-ft. square floating-pan.	Mean monthly temperature (from Table 17), in degrees, Fahrenheit.	Degree—Days: (56.5° × 31, 58.8° × 28, 64.0° × 31, etc.).	
	104%	100%	(*A*)*		For month.	Cumulative (*B*)*
	Inches.	Inches.	Inches.			
January.......	2.52	2.42	2.42	56.5	1 750	1 750
February.....	1.72	1.65	4.07	58.8	1 647	3 397
March........	2.62	2.52	6.59	64.0	1 983	5 380
April..........	3.07	2.95	9.54	67.0	2 010	7 390
May...........	4.70	4.52	14.06	73.3	2 272	9 662
June.........	7.96	7.65	21.71	84.2	2 524	12 186
July...........	7.33	7.04	28.75	84.2	2 609	14 795
August........	8.58	8.25	37.00	86.3	2 674	17 469
September....	6.69	6.43	43.43	84.2	2 524	19 993
October.......	2.86	2.75	46.18	67.8	2 102	22 095
November....	1.51	1.45	47.63	55.0	1 650	23 745
December.....	1.36	1.31	48.94	49.0	1 519	25 264
Year.....	50.92	48.94	48.94	69.2	25 264	25 264

* The Austin curves of Plate XXXIII are plotted from the values, (*A*) and (*B*), of Table 22.

It is noticeable that all the curves of Plate XXXIII (which show the relations between the cumulative evaporation depths in inches and the corresponding "degree-days") are of a similar "ogee" form at all the measured stations, and for a whole year do not depart greatly from a straight line. The departures from the straight line, showing greater rates of evaporation per degree-day for some months than for others, probably are due to other conditions than temperature which affect evaporation, such as the more frequent, more sustained, and more severe winds of some months, the lower humidity of the air during some months, etc., etc.

From Figs. 6 to 10, Fig. 12, and Plates XXXII to XXXIV, and the discussions relating to them, the conclusion seems warranted that (at least for the Great Plateau and for the complete cycle, averaging a year) the monthly depth of evaporation increases with the mean monthly temperature, usually by an approximate "straight-line" relation.

Subsidiary Conclusion (*e*):

Relations Between Evaporation Depth and Elevation Above Sea.

The relations between elevation and evaporation depth are shown and studied on the combined diagrams of Plates XXXII to XXXIV and Fig. 13; but most plainly by Diagram II of Fig. 13, and by Fig. 4 (which was not used in the investigation, but was made only for this paper).

Since, at elevations above sea level, water boils at temperatures below 212° Fahr., it appears at least reasonable that at any given temperature water will vaporize or evaporate more quickly at high than at low elevations.

On Diagram I of Fig. 13 are plotted the curves showing the relations between temperature and evaporation in 3-ft. square floating-pans at five measured evaporation stations on the Great Plateau. On each curve is marked the name and elevation of its station; and this diagram shows in a general way that at equal mean monthly temperatures the evaporation depth is greater at stations of higher elevation. The combined diagram of the Piche evaporimeter observations (Plate XXXII) also shows this in a general way.

However, the increases of evaporation depth with increases both of mean temperature and of elevation are best shown by Diagram II of Fig. 13 (made up from Diagram I), by Tables 23, 24, and 25 (made up by reading intersection values from Diagram II, Fig. 13), and by Fig. 4 (plotted from the values of Tables 23 and 25).

In the study made for Fig. 4, the values of Table 24 were extended upward to Elevation 7 000 ft., downward to 500 ft. below sea level (say as for Salton Sea and Dead Sea), and to the 2-in. increment of 16 to 18 in. monthly evaporation depth; and the values of Table 23

TABLE 23.—MEAN MONTHLY TEMPERATURES CAUSING STATED DEPTHS OF MONTHLY EVAPORATION FROM 3-FT. SQUARE FLOATING-PANS, AT VARIOUS ELEVATIONS ABOVE SEA LEVEL: FOR STATIONS ON GREAT PLATEAU ONLY.

(Values are read from curves of Diagram II, Fig. 13.)

Elevation above sea level, in feet.	MONTHLY DEPTHS OF EVAPORATION.					
	2 in.	4 in.	6 in.	8 in.	10 in.	12 in.
	Monthly Mean Temperature, in Degrees, Fahrenheit.					
7 000	*(26.5°)	(34.7°)	(44.9°)	(55.6°)	(67.3°)	(79.7°)
6 000	(32.7)	(41.7)	(51.5)	(61.6)	(72.0)	(82.9)
5 000	38.8	48.4	57.9	67.5	76.9	(86.3)
4 000	44.7	54.8	64.0	73.1	†82.8 (81.3°)	(89.1)
3 000	50.3	60.7	69.6	78.1	†88.0 (85.2°)	(91.4)
2 000	55.2	66.0	74.6	82.4	(88.4)	(93.2)
1 000	59.1	70.2	78.6	85.4	(90.4)	(93.6)
600	60.4	71.6	79.8	86.3	(90.8)	(93.5)
000	(61.8)	(73.1)	(81.1)	(87.3)	(91.2)	(93.0)

* No actual evaporation data were available above Elevation 5 000 ft., below Elevation 600 ft., or for monthly evaporation depths greater than 10 in.; and all values in Table 23 beyond those limits (shown in parentheses, as (26.5°), etc.) were filled in by use of the supposititious increments of Table 24.

† From the study embodied in Table 24, the observed values in Table 23 for elevations of 4 000 and 3 000 ft. and for 10 in. evaporation (†82.8° and †88.0°) appeared to be abnormal, and hence were replaced (in Tables 23, 24, and 25, and in Fig. 4) by the adjusted supposititious values (81.3°) and (85.2°).

TABLE 24.—INCREMENTS IN MEAN MONTHLY TEMPERATURES CORRESPONDING TO 2-IN. INCREMENTS IN DEPTH OF MONTHLY EVAPORATION FROM 3-FT. SQUARE FLOATING-PANS, ON THE GREAT PLATEAU.

Elevation above sea level, in feet.	2-IN. INCREMENTS IN MONTHLY EVAPORATIONS.				
	2 to 4 in.	4 to 6 in.	6 to 8 in.	8 to 10 in.	10 to 12 in.
	Corresponding Increments in Monthly Mean Temperatures, in Degrees, Fahrenheit.				
7 000	*(8.2°)	(10.2°)	(10.7°)	(11.7°)	(12.4°)
6 000	(9.0)	(9.8)	(10.1)	(10.4)	(10.9)
5 000	9.6	9.5	9.6	9.4	(9.4)
4 000	10.1	9.2	9.1	†9.7 (8.2°)	(7.8)
3 000	10.4	8.9	8.5	†9.9 (7.1°)	(6.2)
2 000	10.8	8.6	7.8	(6.0)	(4.8)
1 000	11.1	8.4	6.8	(5.0)	(3.2)
600	11.2	8.2	6.5	(4.5)	(2.7)
000	(11.3)	(8.0)	(6.2)	(3.9)	(1.8)

* All values in this table not in parentheses are first differences from the corresponding values of Table 23. All values in parentheses, as *(8.2°), etc., are deduced by a more or less careful extension or projection of the first differences not in parentheses, by the aid of second and third differences, and studies of the same diagrams.

† As explained in Table 23, the two values, †9.7° and †9.9°, are believed to be abnormal, and hence are replaced in use by the adjusted values (8.2°) and (7.1°).

were extended to the same limits. This was done because it was surmised that the combined influence on evaporation depth of all other conditions except temperature and elevation might be relatively unimportant—perhaps so small as to permit the estimation of approximate evaporation losses directly from Fig. 4, not only for places on the Great Plateau, but even generally throughout the United States.

However, the diagram of the extended Table 23 showed serious contradictions beyond 12 in. monthly evaporation (*e. g.*, at 400 ft. below sea level and 81.4° mean monthly temperature, the 6 and 18-in. lines intersect and give either 6 or 18 in. monthly evaporation), and hence it shows that values derived by extensions so far beyond the observed limits are very unreliable.

Also, comparisons were made of the evaporation losses shown by such extended Fig. 4, with the observed evaporations at Salton Sea (263 ft. below sea level), Lake Tahoe (6 225 ft. elevation), and Coyote or Laguna Seca (240 ft. elevation)—all in California and outside the limits of the Great Plateau—and all such comparisons showed very serious disagreements, and indicated that Tables 23, 24, and 25, and Fig. 4 are not applicable with safety to places outside the limits of the Great Plateau.

In drawing up Fig. 4 from Tables 23 and 25, the diagrams were restricted nearly to the limits of the observed evaporations (from 600 to 5 000 ft. elevation and from 2 to 10 in. monthly evaporation), though it is believed that, for stations on the Great Plateau, the diagrams may be extended with reasonable accuracy to elevations of 7 000 ft., and to monthly evaporation depths (in 3-ft. square floating-pans) of 12 in. In Tables 23, 24, and 25, the values are extended downward to Elevation 000 ft. or sea level, necessarily outside the limits of the Plateau.

Table 23 is plotted as Diagram (*a*), and Table 25 as Diagram (*b*), of Fig. 4. Table 25 is derived by reading intersection values from Diagram II of Fig. 13 and from the (extended) Diagram (*a*) of Fig. 4. Most of the explanations given of Tables 23 and 24 apply also to Table 25. Some parts of Diagram (*b*) of Fig. 4 are not in close agreement with Diagram (*a*), because of slight graphical adjustment to straight-line relations.

In connection with the discussion of Fig. 4, reference should be made to the yearly evaporation depth shown at the upper edge of Diagram (*b*). The mean temperature for the year is the average of the twelve mean monthly temperatures, and the evaporation temperature relations of Diagram (*b*) are quite generally "straight-line" relations. Hence, if the temperature lines of that diagram are used as yearly mean temperatures and the yearly evaporation depth is read off at the top of the diagram (where the scale is one-twelfth that at the bottom), then approximately the same yearly evaporation will be obtained as if the temperature lines are used as monthly mean tem-

peratures, the monthly evaporation depths read off at the bottom of the diagram, and their sum for the 12 months taken.

TABLE 25.—ELEVATIONS CORRESPONDING TO STATED MEAN MONTHLY TEMPERATURES AND STATED MONTHLY DEPTHS OF EVAPORATION: FOR STATIONS ON GREAT PLATEAU ONLY.

Mean monthly temperatures, in degrees, Fahrenheit.	Monthly Depths of Evaporation from 3-ft. square Floating-Pans.					
	2 in.	4 in.	6 in.	8 in.	10 in.	12 in.
	Elevation, above Sea Level, in Feet.					
30	*(6 410)	....	...	....	....	....
35	(5 610)	(6 960)	....	....	....	....
40	4 800	(6 240)	....	....	....	....
45	3 940	(5 500)	(6 980)	....	....	....
50	3 040	4 740	(6 210)	....	....	....
55	2 030	3 970	(5 460)	(7 090)	....	....
60	710	3 120	4 660	(6 270)	....	....
65	....	2 200	3 820	(5 420)	(7 500)	....
70	....	1 060	2 920	4 570	(6 430)	....
75	....	....	1 910	3 640	(5 390)	(8 460)
80	....	....	(510)	2 560	† 4 500 (4 310)	(6 910)
85	....	....	....	1 160	† 3 600 (3 090)	(5 380)
90	....	....	....	....	(1 300)	(3 660)

* and † See explanations following Tables 23 and 24.

TABLE 26.—SMALL VARIATIONS IN YEARLY MEAN TEMPERATURES, IN DEGREES, FAHRENHEIT.

Station and period.	Mean Yearly Temperatures.		
	Average mean yearly, of all years.	Maximum mean yearly, of all years.	Minimum mean yearly, of all years.
* Salton, Cal.; 12 years (1889-1900)......	76.9° = 100.%	79.7° = + 3.5%	73.3° = − 4.6%
† San José, " ; 26 " (1874-1899)......	58.1 = 100.	61.7 = + 6.2	55.2 = − 4.9
‡ Truckee, " ; 30 " (1871-1900)......	43.9 = 100.	48.8 = + 11.1	37.3 = − 15.0
Average of the three stations..........	100.%	+ 6.9%	− 8.2%

* At Salton Sea, 263 ft. below sea level.

† 12 miles from Coyote or Laguna Seca, and 100 ft. above sea level.

‡ 12 miles from Lake Tahoe, and 5 818 ft. above sea level.

In closing the discussion of Fig. 4, attention should be called to the comparatively small differences at any station in the yearly mean temperatures of different years, the variations being much smaller than those of the rainfall and the stream flow. This comparative constancy of yearly mean temperatures adds greatly to the reliability of estimates of evaporation made by their aid, as by Fig. 4. The small variation in the yearly mean temperature of different years is

shown by the three instances, Tables 26, 27, and 28, worked up in connection with the test mentioned of the attempted general application of Fig. 4.

Tables 26 and 27 are derived from data given by Mr. Alex. G. McAdie.*

Even the monthly mean temperatures (though less constant for different years than the mean yearly) are quite constant in comparison with rainfall and stream flow. This is shown by Table 27.

TABLE 27.—COMPARATIVELY SMALL VARIATIONS IN MONTHLY MEAN TEMPERATURES, IN DEGREES, FAHRENHEIT.

Station.	Period, in years.	Month.	Mean Monthly Temperatures, in Degrees, Fahrenheit.		
			Average mean monthly, of all years.	Maximum mean monthly, of all years.	Minimum mean monthly, of all years.
Salton, Cal....	12	Jan......	55.7° = 100.%	65.7° = + 8.0%	49.1° = − 11.7%
		Feb......	58.8 = 100.	67.9 = + 15.6	49.6 = − 15.6
		Mar......	66.0 = 100.	74.0 = + 12.2	57.8 = − 12.3
		Apr......	76.5 = 100.	82.1 = + 7.3	70.3 = − 8.0
		May......	83.1 = 100.	94.0 = + 13.0	73.3 = − 11.6
		June.....	93.8 = 100.	100.6 = + 7.2	86.9 = − 7.4
		July.....	98.9 = 100.	107.0 = + 8.2	94.6 = − 4.3
		Aug......	97.2 = 100.	107.4 = + 10.6	89.3 = − 8.2
		Sept.....	91.0 = 100.	99.9 = + 11.0	85.8 = − 5.6
		Oct......	79.1 = 100.	85.3 = + 7.9	72.6 = − 8.2
		Nov......	66.8 = 100.	70.8 = + 6.0	59.1 = − 11.5
		Dec......	56.1 = 100.	66.3 = + 18.2	46.2 = − 17.6
		Average.	76.9° = (100.%)	85.1° = (+ 10.4%)	69.6° = (− 10.2%)
San José, Cal..	26	Jan......	48.2° = 100.%	57.7° = + 20.0%	40.4° = − 16.0%
		Feb......	50.7 = 100.	54.8 = + 8.0	45.3 = − 10.6
		Mar......	53.6 = 100.	57.8 = + 7.8	48.5 = − 9.4
		Apr......	56.4 = 100.	62.1 = + 10.2	51.9 = − 8.0
		May......	60.1 = 100.	67.7 = + 12.8	50.1 = − 16.5
		June.....	65.5 = 100.	76.1 = + 16.1	60.5 = − 7.6
		July.....	66.7 = 100.	71.1 = + 6.9	64.7 = − 3.0
		Aug......	66.4 = 100.	70.1 = + 5.8	63.3 = − 4.7
		Sept.....	64.7 = 100.	71.0 = + 9.8	62.0 = − 4.0
		Oct......	60.2 = 100.	65.5 = + 9.0	56.3 = − 6.5
		Nov......	54.1 = 100.	57.6 = + 6.6	48.5 = − 10.2
		Dec......	50.1 = 100.	57.9 = + 15.7	46.1 = − 8.0
		Average.	58.1° = (100.%)	64.1° = (+ 10.7%)	53.1° = (− 8.7%)
Truckee, Cal...	30	Jan......	25.3° = 100.%	32.9° = + 30.0%	16.4° = − 35.2%
		Feb......	28.3 = 100.	39.0 = + 37.7	21.4 = − 24.5
		Mar......	32.9 = 100.	42.0 = + 27.9	25.7 = − 21.8
		Apr......	40.0 = 100.	50.3 = + 25.8	25.7 = − 35.7
		May......	48.4 = 100.	57.7 = + 19.3	37.6 = − 22.3
		June.....	57.4 = 100.	70.5 = + 23.0	48.2 = − 16.0
		July.....	65.4 = 100.	73.1 = + 12.0	53.0 = − 19.0
		Aug......	63.4 = 100.	73.2 = + 15.6	52.5 = − 17.0
		Sept.....	55.9 = 100.	61.6 = + 31.0	50.5 = − 9.6
		Oct......	45.1 = 100.	60.6 = + 12.1	38.7 = − 14.2
		Nov......	36.5 = 100.	43.4 = + 19.0	29.0 = − 20.5
		Dec......	28.7 = 100.	35.9 = + 25.0	21.5 = − 25.0
		Average.	43.9° = (100.%)	52.5° = (+ 23.2%)	35.0° = (− 21.7%)

* In "Climatology of California", 1903, *Bulletin L* of the Weather Bureau, U. S. Dept. of Agriculture.

From Table 26, at Salton, San José, and Truckee, the maximum departure from the yearly mean temperature is only 15% and the average departure only about 8%; and from Table 27, the maximum departure from the monthly mean temperature is only about 38% and the average departure less than half that. In comparison, the extremes of rainfall and stream flow at any place usually are about two and one-half times the mean values.

Not only is the yearly mean temperature at any place reasonably constant, but the yearly evaporation depth also is quite constant from year to year, much more so than the rainfall or the stream flow. Without going into this question thoroughly, some data to confirm this view are given in Tables 28 and 29.

TABLE 28.—Comparatively Close Agreement of Yearly Evaporation Depths in Santa Clara Valley, California, in Two Different Years: Measured in 3-Ft. Square Pans.*

Group and No. of Pan.	Year, 1904.		Year, 1905.			
	Year's evaporation depth.		Year's evaporation depth.		Departure from year, 1904.	
	Inches.	Percentage.	Inches.	Percentage.	—	+
Coyote Valley Group.						
Pan No. 1 (land-pan)........ ..	49.8	100	46.7	93.9	6.1%	......
" " 2 "	49.7	100	45.8	92.3	7.7	......
Laguna Seca Group.						
Pan No. 3 (land-pan).........	53.8	100	55.4	103.0	....	3.0%
" " 4 "	62.3	100	60.4	97.0	3.0%	......
" " 5 "	53.8	100	54.8	101.8	....	1.8
" " 6 (floating-pan)......	39.4	100	43.1	109.4	....	9.4
" " 7 "	43.0	100	53.5	124.5	....	24.5
Upper Gorge Group.						
Pan No. 8 (land-pan).........	41.7	100	52.3	125.5	....	25.5
" " 9 "	47.2	100	45.2	95.9	4.1	......
" " 10 "	46.5	100	55.4	119.0	....	19.0
" " 11 (floating-pan)......	34.3	100	30.4	88.7	11.3	......
Averages........................	....	100	....	104.7	— 6.4%	+13.9%
Extremes........................	....	....	....	....	—11.3	+25.5

* Data from *Engineering News*, Vol. 67, pp. 380–383, February 29th, 1912. "California Evaporation Records", by Edwin Duryea, Jr., M. Am. Soc. C. E.

From Tables 28 and 29, respectively, the averages of the variations in the yearly evaporation depth are about 10 and 6%, and the maximum variations about 25 and 14 per cent.

Finally, quoting from Folwell*:

"Evaporation is seen to vary less than does rainfall, the greatest variation from the annual mean at Boston during sixteen years, for instance, being about 13%."

* "Water-Supply Engineering", 1906, p. 90.

TABLE 29.—SMALL VARIATIONS OF THE YEAR'S EVAPORATION DEPTH FROM YEAR TO YEAR, AS OBSERVED FOR SEVERAL SUCCESSIVE YEARS.

At Lake Tahoe, California, as Measured in a 2-Ft. Square Floating-Pan.*					At Emdrup, Denmark (no Data of Pan).†				
Years (6).	Year's evaporation depth.		Departure from average mean yearly depth.		Years (11).	Year's evaporation depth.		Departure from average mean yearly depth.	
	Inches.	Percentage.	−	+		Inches.	Percentage.	−	+
1900-01	28.2	92.0	8.02%	....	1849	29.5	105.7	....	5.7%
1901-02	25.7	84.0	16.0	....	1850	29.1	104.2	....	4.2
1902-03	33.0	107.9	....	7.9%	1851	28.4	101.8	....	1.8
1903-04	31.3	102.2	....	2.2	1852	29.4	105.3	....	5.3
1904-05	33.9	110.8	....	10.8	1853	26.9	96.3	3.7%	....
1905-06	31.4	102.6	....	2.6	1854	27.9	100.0	0.0	0.0
					1855	25.1	89.9	10.1	....
					1856	24.0	86.0	14.0	....
					1857	29.9	107.2	....	7.2
					1858	30.6	108.1	....	8.1
					1859	26.4	94.5	5.5	....
Average...	30.6	100.0	−12.0%	+ 5.9%	Average...	27.9	100.0	−6.7%	+4.6%
Extremes..	....	.....	−16.0	+10.8	Extremes.	....	.....	−14.0	+8.1

* From *Engineering News*, Vol. 67, pp. 380–383.
† From Fanning's "Water-Supply Engineering", 1906, p. 89.

This closes the discussion and working out of the five subsidiary conclusions of this investigation. They are stated briefly and precisely in Table 2.

There remains the working out of the principal conclusion, the estimation of the yearly evaporation depth from Lake Conchos by five more or less independent methods.

PRINCIPAL CONCLUSION:

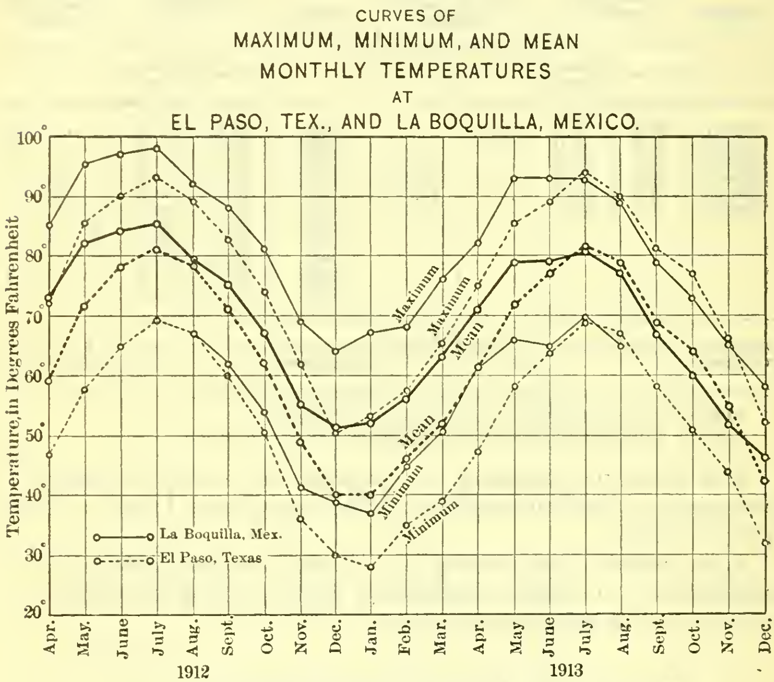
YEARLY DEPTH OF EVAPORATION FROM LAKE CONCHOS.

Method A.—This method consists of estimating the evaporation depth from Lake Conchos by means of Diagram II of Fig. 13 (the evaporation-temperature-elevation relations established for the stations observed in Texas and New Mexico), used in conjunction with the observed mean monthly temperatures at Lake Conchos.

The elevation of Lake Conchos is 4 320 ft. above sea level. On Diagram II of Fig. 13 is drawn a line at that elevation from which were read off the mean monthly temperatures necessary at that elevation to cause monthly evaporation depths (from 3-ft. square floating-pans) of 2, 4, 6, 8, and 10 in. Those five values of mean monthly temperature are plotted on Fig. 15, where it is seen that they conform almost exactly to a straight line. This straight line (which represents

the monthly evaporation depths corresponding to all monthly mean temperatures at places on the Great Plateau in Texas and New Mexico having elevations of 4 320 ft.) it is assumed may be applied safely to Lake Conchos; also, on the Great Plateau, but about 500 miles farther south, in Mexico.

In Table 37 and on Fig. 14 are given the observed monthly mean temperatures (the mean of the daily mean temperatures for each



Note:-Later data give the following additional temperatures at La Boquilla

Jan. 1914-Max., 69°; Min., 55°; Mean, 62°.
Feb. 1914- " 73°; " 40°; " 56°

The corresponding temperatures, at El Paso were

Jan. 1914-Max., 61°; Min., 35°; Mean, 48°
Feb. 1914- " 61.5°; " 36.5°; " 49°

FIG. 14.

month) at La Boquilla for 21 months, April, 1912, to December, 1913, inclusive. In Table 30 these twenty-one monthly mean temperatures are repeated (with that for January, 1914, added), and there are given also the twenty-two corresponding depths of monthly evaporation, as read off from Fig. 15.

From Table 30, the evaporation depth from 3-ft. square pans floating on Lake Conchos should have been 80.43 in. for the calendar year,

1913; but, for a composite year, made up by adding the mean evaporations for each of the 12 months, it would be 85.59 in., or 6.5% greater.

From the 22 months' record, however, it is practicable to make up eleven periods, of 12 consecutive months each, and so gain much more knowledge of the variations in yearly evaporation depth. This is done in Table 31.

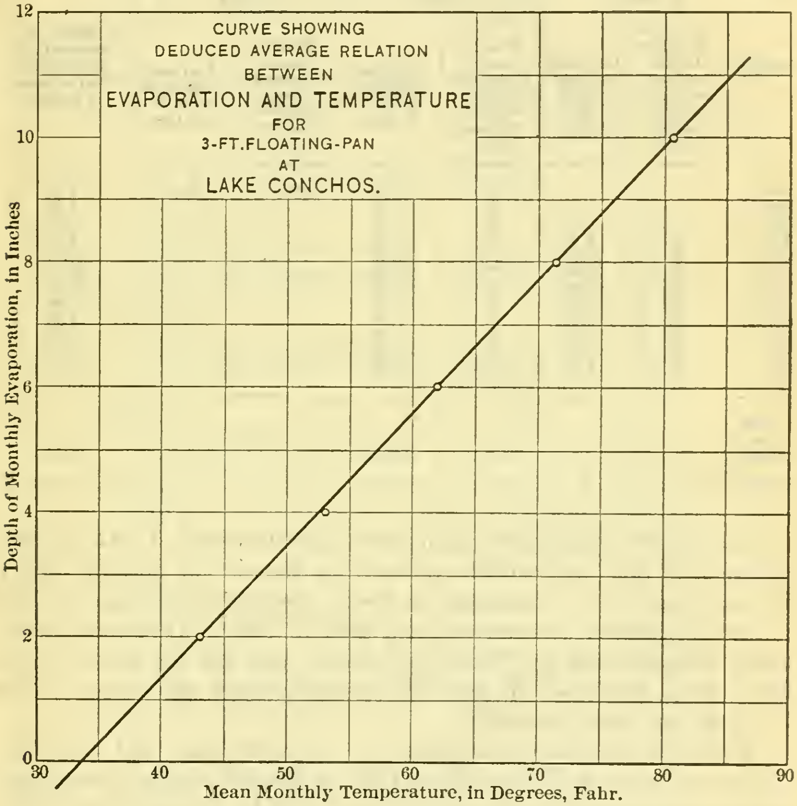


FIG. 15.

The variations in the evaporation depths for 12-month periods are shown better by Table 32.

It is believed that the most reliable and accurate value from Method *A* of the yearly evaporation depth from 3-ft. square pans floating on Lake Conchos is (so far as can be obtained from the data) 84.74 in.

By Subsidiary Conclusion (*c*), the yearly evaporation depth from a lake surface is only 62% of that from a 3-ft. square pan floating

thereon;* hence the corresponding yearly evaporation depths from Lake Conchos itself, by Method *A*, would be as given in Table 33.

TABLE 30.—OBSERVED MONTHLY MEAN TEMPERATURES; AND CORRESPONDING DEPTHS OF EVAPORATION, AS DEDUCED FOR 3-FT. SQUARE FLOATING-PANS, AT LAKE CONCHOS.

Month.	1912.		1913.		1914.		Mean of observations by months. Evaporation, in inches.
	Monthly mean temperature, in degrees, Fahrenheit.	Deduced evaporation, in inches.	Monthly mean temperature, in degrees, Fahrenheit.	Deduced evaporation, in inches.	Monthly mean temperature, in degrees, Fahrenheit.	Deduced evaporation, in inches.	
Jan....	......	......	52	3.90	62.0	6.00	4.95
Feb....	......	......	56	4.75	......	......	4.75
Mar....	......	......	63	6.23	......	......	6.23
Apr....	73	8.34	71	7.91	......	......	8.13
May...	82	10.24	79	9.61	......	......	9.93
June...	84	10.65	79	9.61	.....	......	10.13
July....	85	10.86	81	10.03	......	......	10.44
Aug...	79	9.61	77	9.18	......	......	9.40
Sept...	75	8.75	67	7.07	......	......	7.91
Oct....	67	7.07	60	5.59	......	......	6.33
Nov....	55	4.54	52	3.90	......	......	4.22
Dec....	51	3.69	46	2.65	......	......	3.17
Total or mean.	......	......	......	80.43	......	......	85.59

The yearly evaporation depth from Lake Conchos of 52.5 in. was accepted as the most reliable value from Method *A*; but, for safety in providing fully for evaporation losses, the maximum value of 55.0 in. (4.7% greater) was adopted and used in March, 1914—and subsequent investigations by Methods *B* and *C*, and for the period up to June, 1914, developed no need for changing from the value of 55.0 in. (1.40 m.) then adopted.

Method B.—This method consists of the modification and correction of the result arrived at by Method *A*, by the aid of the short-period pan measurements of evaporation made at Lake Conchos, first for 2 and later for 5 months. Two variants of Method *B* were used, B_a and B_b; and a third variant B_0 was attempted, but was rejected as unreliable.

The rejected method B_0 consisted of a direct percentage comparison of simultaneous observations of pan evaporations at El Paso and at Lake Conchos for the same short period (16 days), used in conjunction with the observed yearly evaporation depth at El Paso.

* See Table 2.

TABLE 31.—ESTIMATED DEPTH OF EVAPORATION AT LAKE CONCHOS FOR ELEVEN 12-MONTH PERIODS.
From Evaporation-Temperature Study.

Month.	Estimated Depths of Evaporation by 12-month Periods (in 3-Ft. Square Floating-Pans).											Means.	
	Apr., 1912, to Mar., 1913, inclusive.	May, 1912, to Apr., 1913, inclusive.	June, 1912, to May, 1913, inclusive.	July, 1912, to June, 1913, inclusive.	Aug., 1912, to July, 1913, inclusive.	Sept., 1912, to Aug., 1913, inclusive.	Oct., 1912, to Sept., 1913, inclusive.	Nov., 1912, to Oct., 1913, inclusive.	Dec., 1912, to Nov., 1913, inclusive.	Jan., 1913, to Dec., 1913, inclusive.	Feb., 1913, to Jan., 1914, inclusive.	In inches.	Percentage of distribution.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.		
January...	3.90	3.90	3.90	3.90	3.90	3.90	3.90	3.90	3.90	3.90	6.00	4.09	4.8%
February..	4.75	4.75	4.75	4.75	4.75	4.75	4.75	4.75	4.75	4.75	4.75	4.75	5.6
March.....	6.23	6.23	6.23	6.23	6.23	6.23	6.23	6.23	6.23	6.23	6.23	6.23	7.4
April......	8.34	7.91	7.91	7.91	7.91	7.91	7.91	7.91	7.91	7.91	7.91	7.95	9.4
May.......	10.24	10.24	9.61	9.61	9.61	9.61	9.61	9.61	9.61	9.61	9.61	9.72	11.5
June......	10.65	10.65	10.65	9.61	9.61	9.61	9.61	9.61	9.61	9.61	9.61	9.90	11.7
July.......	10.86	10.86	10.86	10.86	10.03	10.03	10.03	10.03	10.03	10.03	10.03	10.33	12.2
August....	9.61	9.61	9.61	9.61	9.61	9.18	9.18	9.18	9.18	9.18	9.18	9.38	11.1
September	8.75	8.75	8.75	8.75	8.75	8.75	7.07	7.07	7.07	7.07	7.07	7.99	9.4
October...	7.07	7.07	7.07	7.07	7.07	7.07	7.07	5.59	5.59	5.59	5.59	6.53	7.7
November.	4.54	4.54	4.54	4.54	4.54	4.54	4.54	4.54	3.90	3.90	3.90	4.37	5.1
December.	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	2.65	2.65	3.50	4.1
Totals.....	88.63	88.20	87.57	86.53	85.70	85.27	83.59	82.11	81.47	80.43	82.53	84.74	100%

TABLE 32.—VARIATIONS IN YEARLY EVAPORATION DEPTH.

12-month period, or year.	Corresponding Depth of Evaporation from 3-ft. Square Pans Floating on Lake Conchos.	
	In inches.	Percentages.
April, 1912, to March, 1913, inclusive	88.63	104.7
May, " " April, " "	88.20	104.0
June, " " May, " "	87.57	103.3
July, " " June, " "	86.53	102.0
August, " " July, " "	85.70	101.0
September, " " August, " "	85.27	100.6
October, " " September, " "	83.59	98.6
November, " " October, " "	82.11	96.9
December, " " November, " "	81.47	96.0
January, 1913, " December, " "	80.43	94.9
February, " " January, 1914, "	82.53	97.4
Average for 11 periods	84.74	100
Extremes for 11 periods	88.63 and 80.43	+ 4.7 and − 5.1
Corresponding composite value from Table 31	85.59	101.0

TABLE 33.—YEARLY EVAPORATION DEPTHS FROM LAKE CONCHOS, BY METHOD *A*.

	Pan evaporation.	Lake evaporation.
Maximum value	88.63 × 0.62	= 55.0 in. = 104.7%
Minimum value	80.43 × "	= 49.8 " = 94.9
Value for calendar year, 1913	80.43 × "	= 49.8 " = 94.9
Value for composite year, all mean monthly temperatures	85.59 × "	= 53.0 " = 101.0
Value as average for the eleven 12-month periods	84.74 × "	= 52.5 " = 100.0

The observed pan evaporations and mean temperatures at El Paso, and at La Boquilla, for January-March, 1914, are given in full in Table 34 (Plate XXXV), and are summarized in Table 35.

The observations at El Paso for January and February were rejected, because they covered only 3 days in January and 2 days in February. For various reasons, including shooting of the pans by Mexicans and cowboys, it was found impracticable to carry out a fairly unbroken series of evaporation observations at El Paso within the time available.

As shown by Table 35, the deduced evaporation depths at El Paso from 3-ft. square floating-pans for the entire month of March were 2.43 in., as measured in Pan No. 1, and 2.53 in., as measured in Pan No. 2, or a mean depth of 2.48 in.; the corresponding evaporation depths at Lake Conchos for March were 4.40, 5.12, 5.44, and 5.74 in., as measured in four pans, or a mean of 5.17 in. Hence the evaporation

TABLE 34.—MEASURED EVAPORATIONS AND TEMPERATURES AT LA BOQUILLA, MEXICO, AND EL PASO, TEX., FOR JANUARY, FEBRUARY, AND MARCH, 1914.

January

Date 1914 January	At La Boquilla, Mexico: Evaporation, in inches: Land pans No. 1	Land pans No. 3	Floating pans No. 2	Floating pans No. 4	Temperature: Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.	Weather conditions at La Boquilla	El Paso, Texas: Evaporation, in inches: Land pans No. 1	No. 2	No. 3	Temperature: Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.
1					71							60	42	55
2					71							62	39	51
3					71							58	35	51
4					71							58	36	47
5					71							56	31	45
6					66							54	31	46
7					61							62	33	47
8					71							62	38	47
9					66							66	43	55
10					56							54	38	46
11					59							45	33	40
12					61			Clear bright day				50	35	43
13	6/32				67			Clear bright day				59	35	47
14	4/32	4/32			71	63	67	Clear bright day				69	25	45
15	8/32	10/32			75	64	69	Clear bright day, warm				71	31	51
16	7/32	8/32			73	59	68	Morning clear cloudy afternoon				64	37	50
17	9/32	6/32	2/32		76	66	72	Morning cloudy, bright, clear afternoon				74	41	57
18	6/32	6/32	4/32		72	62	67	Clear and bright, stiff breeze				65	40	51
19	10/32	8/32	6/32		73	61	64	Clear and bright, moderate breeze				62	40	51
20	6/32	4/32	*		67	54	63	Cloudy, high wind				67	37	52
21	6/32	6/32	*		61	55	59	Morning cloudy, afternoon clear, no wind				58	34	46
22	6/32	6/32	*		67	57	62	Clear and bright, no wind				60	38	50
23	6/32	4/32	4/32		72	51	66	Cloudy, high wind				54	35	45
24	14/32	8/32	6/32		77	61	69	Bright and clear, calm				61	31	46
25	8/32	6/32	6/32		69	63	65	Bright and clear, calm				56	27	42
26	6/32	8/32	8/32		79	64	70	Clear, light breeze				55	31	43
27	10/32	6/32	*	6/32	76	56	71	Bright and clear, light breeze	1/32	1/32	1/32	58	41	50
28	10/32	16/32	*	*	79	49	64	Bright and clear, very high wind	1/32	1/32	2/32	51	38	45
29	14/32	12/32	*	*	57	29	45	Bright and clear, high wind	1/32	2/32	3/32	44	31	38
30	4/32	6/32	4/32	6/32	54	30	42	Bright and clear, light wind	(Discontinued.)			49	31	40
31	4/32	4/32	4/32		65	49	57	Bright and clear, calm				59	35	42
Total or average	4.87	4.04	1.44	0.41	69	55	64		0.37	0.34	0.31	61.4	35.4	48.3

* Submerged.

February

Date 1914 February	At La Boquilla, Mexico: Evaporation, in inches: Land pans No. 1	Land pans No. 3	Floating pans No. 2	Floating pans No. 4	Temperature: Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.	Weather conditions at La Boquilla	El Paso, Texas: Evaporation, in inches: Land pans No. 1	No. 2	No. 3	Temperature: Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.
1	4/32	4/32	5/32	....	71	25	49	Bright and clear, calm	...	...	(Discontinued.)	67	30	48
2	5/32	3/32	16/32	....	74	29	52	Morning cloudy; afternoon bright, clear	...	...		64	40	53
3	6/32	4/32	6/32	....	79	40	59	Bright and clear, perceptible breeze	...	...		64	30	52
4		6/32	6/32	....	77	40	58	Morning bright, clear; afternoon cloudy	...	...		63	43	53
5	2/32	†	*	....	73	45	60	Bright, clear, very high wind	...	...		65	42	54
6	10/32	†	*	....	73	52	63	A. M. cloudy, afternoon bright and clear, very high wind	...	...		58	30	48
7	7/32	†	*	....	73	38	56	Cloudy, light breeze	...	...		57	29	43
8	5/32	6/32	6/32	1/32	73	45	54	Cloudy, light breeze	...	...		59	31	46
9	4/32	4/32	6/32	1/32	72	37	55	Bright and clear, perceptible breeze	...	...		65	30	47
10	6/32	6/32	4/32	4/32	73	40	57	Bright and clear, perceptible breeze	...	...		67	35	51
11	8/32	6/32	4/32	4/32	71	46	64	Morning bright, clear; afternoon cloudy	...	...		71	36	54
12	8/32	6/32	4/32	....	73	45	60	Morning cloudy; afternoon bright and clear, high wind		...		68	48	58
13	6/32	4/32	7/32	*	75	41	58	Morning bright and clear; afternoon cloudy, light wind		...		60	35	48
14	6/32	4/32	7/32	*	74	42	58	Bright and clear, high wind				61	33	47
15	6/32	6/32	4/32	*	68	40	54	Bright, clear, moderate breeze				58	33	48
16	6/32	6/32	4/32	*	79	45	57	Cloudy, high wind				60	40	50
17	5/32	6/32	4/32	*	71	48	60	Cloudy, 1.6 mm. rain, perceptible breeze				59	42	51
18	3/32	4/32	3/32	*	72	40	58	Bright and clear, high wind				59	45	52
19	2/32	3/32	5/32	*	78	44	58	Bright and clear, light wind				63	45	54
20	6/32	6/32	6/32	*	71	45	57	Bright and clear, light wind				67	32	51
21	7/32	6/32	6/32	*	80	38	59	Bright and clear, perceptible wind				72	40	56
22	7/32	5/32	6/32	2/32	74	47	58	Cloudy, very high wind				55	42	48
23	7/32	6/32	6/32	2/32	71	44	60	Morning cloudy, afternoon bright, high wind				57	35	46
24	7/32	7/32	*		73	22	51	Morning cloudy; afternoon bright, high wind				42	26	34
25	8/32	6/32	*		65	22	41	Bright and clear, light wind				54	26	40
26	7/32	7/32	*		69	22	50	Bright and clear, moderate breeze				64	22	43
27	7/32	....	8/32	*	73	48	58	Morning bright and clear; afternoon cloudy, high wind	2/32	4/32		54	32	43
28	6/32	....	7/32	*	65	30	47	Morning bright and clear; moderate breeze	2/32	2/32		66	34	50
Total or average	5.1	5.5	4.2	0.63	72	40	56		0.12	0.19		61.5	36.5	49

† Pan being reset. * Submerged.

March

Date 1914 March	At La Boquilla, Mexico: Evaporation, in inches: Land pans No. 1	Land pans No. 3	Floating pans No. 2	Floating pans No. 4	Temperature: Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.	Weather conditions at La Boquilla	El Paso, Texas: Evaporation, in inches: Land pans No. 1	No. 2	No. 3	Temperature: Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.
1	6/32	6/32	7/32	.	76	34	55	Bright and clear, brisk breeze	4/32	6/32	(Discontinued.)	72	36	54
2	5/32	7/32	6/32		76	36	56	Bright and clear, stiff breeze	2/32	2/32		72	42	57
3	6/32	8/32	‡		69	50	60	Bright and clear, high wind	2/32	2/32		54	34	44
4	6/32	7/32	‡	*	60	31	45	Bright and clear, stiff wind	2/32	2/32		62	32	47
5	6/32	7/32	‡	*	78	24	51	Bright and clear, squally breeze	1/32	4/32		71	43	57
6	6/32	4/32	‡	*	72	36	55	Bright and clear, high wind	2/32	2/32		67	37	52
7	6/32	6/32	‡	*	65	35	50	Bright and clear, high wind	1/32	4/32		58	36	47
8	6/32	6/32	4/32	6/32	72	34	53	Bright and clear brisk breeze	4/32	4/32		66	31	49
9	6/32	6/32	4/32	6/32	62	42	52	Cloudy, light breeze	1/32	1/32		72	32	55
10	.			.				....	4/32	2/32		74	41	53
11									2/32	1/32		53	36	45
12									4/32	2/32		67	32	44
13									2/32	4/32		54	28	46
14									4/32	1/32		78	35	51
15									2/32	2/32				
16									4/32	4/32				
17														
18														
19														
20														
21														
22														
23														
24														
25														
26														
27														
28														
29														
30														
31														
Total or average	1.50	1.97	0.66	0.37					1.50	1.63				

‡ Pan leaking. * Submerged.

NOTES.

EVAPORATION-PANS AT LA BOQUILLA.

The evaporation pans set at La Boquilla are of galvanized iron, 3 ft. square and 1 ft. deep. A rain gauge was set near each group of pans.

The land-pans were originally set on top of the ground (or rock) surface and a strip of canvas was tacked around the outside of the pans to protect the sun from striking their metal sides. On February 4th the land-pans were reset in earth. Because of the rocky character of the ground it was necessary to haul the soil to the site and deposit it around the pans. Secure fences were provided around the pans as protection from animals, and an observer was commissioned to ride continually back and forth to the pans to prevent any interference with them, observe any leaks or unusual conditions of any nature, and to make readings.

The floating pans were protected from spray by flash boards on the rafts, but their records were frequently interfered with by leaks or waves, and by being blown ashore during high winds.

The description of the location of individual pans follows.

Pan No. 1 (Land-Pan).—Set January 12th, 1914, at 5.00 P. M., about 2 000 ft. up stream from La Boquilla Dam at Elevation 1 295 m. Pan is in path of prevailing winds and gets sun entire day.

Pan No. 2 (Floating Pan).—Set January 14th, 1914, at 4.30 P. M. floating on Lake Conchos near Land-Pan No. 1. Pan is built inside a strong raft which has flash-boards to prevent spray from entering pan.

Pan No. 3 (Land-Pan).—Set January 14th, 1914, at 4 P. M., on a promontory about 4 km. up stream from La Boquilla Dam, at Elevation 1 317 m. Pan is in open stretch of country.

Pan No. 4 (Land-Pan).—Set January 26th, 1914, floating on Lake Conchos near Land-Pan No. 3. Pan is of similar construction, and protected in a similar manner as No. 2, but winds and waves interfered with the record to a great extent.

EVAPORATION-PANS AT EL PASO.

Three floating pans and three land pans 3 ft. square, of the same type as those set at La Boquilla, were set at a cement plant about 8 miles from El Paso, Tex., on January 24th, 1914. The reservoir in which the floating-pans were set was emptied immediately after their installation so that no record was secured. The land pans, after but 5 days' record was secured, were so interfered with by winds, which blew sand into them, that two pans were moved from their original positions into a rear yard near the Weather Bureau Station in El Paso, on February 26th. It is not known to what extent near by buildings or fences may interfere with air currents, or tend toward a reduced evaporation depth at this location.

The El Paso records are not considered satisfactory.

TEMPERATURES.

Temperatures given for La Boquilla are those observed by the Mexican Northern Power Company with maximum and minimum thermometers. During January, 1914, the minimum thermometer was observed to be inexact, and the temperatures here given for that month are those observed at Pilar de Conchos at the up-stream end of the reservoir. The temperatures at Pilar de Conchos are known to conform very closely to those at La Boquilla.

The record of temperature for El Paso here given is that taken by the United States Weather Bureau.

The mean daily temperature as here used is the mean of the maximum and minimum for the day, as is the practice of the United States Weather Bureau. The mean monthly temperature is the average of the mean daily temperatures for the month.

TABLE 35.—OBSERVED PAN EVAPORATIONS AND MEAN TEMPERATURES, JANUARY–MARCH, 1914.

AT LAKE CONCHOS, MEXICO.

No. of evaporation pan.	(Land-pan) No. 1.			(Floating-pan) No. 2.			(Land-pan) No. 3.			(Floating-pan) No. 4.		
For month of (1914)	Jan.	Feb.	Mar.	Jan.	Feb.	Mar.	Jan.	Feb.	Mar.	Jan.	Feb.	Mar.
No. of days measured	19	27	9	9	21	4	18	22	9	2	6	2
Total depth measured, in inches	4.87	5.1	1.60	1.44	4.2	0.66	4.04	3.5	1.97	0.44	0.63	0.37
Average daily evaporation depth, in inches	0.26	0.19	0.18	0.16	0.20	0.16	0.22	0.16	0.22	0.22	0.10	0.19
Corresponding depth for full month, in inches	7.95	5.3	5.5	4.96	5.60	5.12	6.96	4.46	6.8	6.82	2.91	5.74
Depth for 3-ft floating-pan,* in inches	6.36	4.25	4.40	4.96	5.60	5.12	5.57	3.57	5.44	6.82	2.94	5.74
Mean temperature, in degrees, Fahrenheit, for period of measurement.	64	56	54	64.5	56	56	64	56	54	60	58	57
Mean temperature, in degrees, Fahrenheit, for entire month	62	56	......	62	56	......	62	56	......	62	56	......

AT EL PASO, TEX.

No. of evaporation pan.	(Land-pan) No. 1.			(Land-pan) No 2.			(Land-pan) No. 3.		
For month of (1914)	Jan.	Feb.	Mar.	Jan.	Feb.	Mar.	Jan.	Feb.	Mar.
Number of days measured	3	2	16	3	2	16	3	......	......
Total depth measured, in inches	0.37	0.12	1.56	0.34	0.19	1.63	0.31	......	......
Average daily evaporation depth, in inches	0.12	0.06	0.10	0.11	0.10	0.10	0.10	......	......
Corresponding depth for full month, in inches	3.82	1.68	3.03	3.51	2.66	3.16	3.20	......	......
Depth for 3-ft. floating-pan,* in inches	3.05	1.34	2.43	2.81	2.13	2.53	2.56	......	......
Mean temperature, in degrees, Fahrenheit, for period of measurement.	59	28	†51	59	27	†51	59	......	......
Mean temperature, in degrees, Fahrenheit, for entire month	48	49	......	48	49	......	48	......	......

* As 80% of land-pan.

† These mean temperature results cover only 14 days of the period of measurement. Temperatures recorded by U. S. Weather Bureau.

depth at Lake Conchos apparently was $(5.17 \div 2.48) = 2.09$ times that at El Paso.

TABLE 37.—MEAN MONTHLY TEMPERATURES AT EL PASO, TEX., AND LA BOQUILLA, MEXICO.

In Degrees, Fahrenheit.

Station and Month.	El Paso, Tex.			La Boquilla, Mexico.		
	Maximum.	Minimum.	Mean.	Maximum.	Minimum.	Mean.
1912, April	72.0	46.5	59.0	85.0	....	*73.0
May	85.5	57.5	71.5	95.0	....	*82.0
June	90.0	65.0	78.0	97.0	....	*84.0
July	93.0	69.0	81.0	98.0	....	*85.0
August	89.0	67.0	78.0	92.0	67.0	79.0
September	82.5	60.0	71.0	88.0	62.0	75.0
October	74.0	50.5	62.0	81.0	54.0	67.0
November	62.0	36.0	49.0	69.0	41.0	55.0
December	50.0	30.0	40.0	64.0	39.0	51.0
1913, January	53.0	28.0	40.0	67.0	37.0	52.0
February	57.0	35.0	46.0	68.0	45.0	56.0
March	65.0	39.0	52.0	76.0	51.0	63.0
April	75.0	47.0	61.0	82.0	61.0	71.0
May	85.5	58.0	72.0	93.0	66.0	79.0
June	89.0	64.0	77.0	93.0	65.0	79.0
July	94.0	69.0	81.5	93.0	70.0	81.0
August	90.0	67.0	79.0	89.0	65.0	77.0
September	81.0	58.0	69.0	79.0	....	*67.0
October	77.0	51.0	64.0	73.0	....	*60.0
November	66.0	44.0	55.0	65.0	....	*52.0
December	52.0	32.0	42.0	58.0	....	*46.0

NOTE.—The maximum and minimum temperatures given in Table 37 are the means of all the maximum and all the minimum daily observations for each month. The mean temperatures given are the means of all daily means for the month, the latter being the mean of the maximum and minimum daily.

* The mean annual temperatures for 1913 were—for El Paso, 61.5°; for La Boquilla, 65.2°. Mean monthly temperatures at La Boquilla shown thus (*73°) are deduced from observed maximum temperatures for the corresponding months. During these months no minimum temperatures were observed, due to the fact that no recording thermometer for minimum temperatures was on hand. The mean temperatures, therefore, were deduced by two methods: (1) by percentage relations between maximum and mean temperatures during months when measurements were made, and (2) by first deducing minimum temperatures by average daily differences between maximum and minimum temperatures for observed months.

The daily temperatures at La Boquilla were taken from the monthly hydrographic report sheets of the Mexican Northern Power Co., Ltd., and those at El Paso from the U. S. Weather Bureau Climatological Reports.

The observed evaporation depth at El Paso for a year is 82.0 in. (see Table 17, Plate XXXI); hence, on the supposition that the simultaneous observations of March, 1914, represent the true relative evaporation depths at El Paso and Lake Conchos, the yearly evaporation depth at the latter from 3-ft. square floating-pans should be $(82.0 \times 2.09) = 171$ in., or 14.25 ft.

Such an evaporation depth evidently is absurd; and an analysis of the El Paso data in connection with this method of deduction (B_0) shows that the method is unreliable and unsafe when used with short-period observations, for the reason that during the months of small

TABLE 36.—MEASURED EVAPORATIONS AND TEMPERATURES AT LA BOQUILLA, MEXICO, FOR MARCH, APRIL, MAY, AND JUNE, 1914.

Date, 1914. March.	Evaporation, in Inches. Land-pans. No. 1	No. 8	Floating-pans. No. 2.	No. 4.	Temperatures. Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.	Weather conditions.
1	8/32	6/32	7/32	†	79	51	66	Bright and clear, brisk breeze
2	5/32	7/32	6/32	†	76	36	56	Bright and clear, stiff breeze
3	6/32	8/32	*	†	69	50	60	Bright and clear, high wind
4	6/32	7/32	*	‡	60	31	45	Bright and clear, stiff wind
5	6/32	7/32	*	‡	73	29	51	Bright and clear, squally breeze
6	6/32	8/32	*	‡	72	38	55	Bright and clear, high wind
7	8/32	8/32	*	‡	68	38	53	Bright and clear, high wind
8	5/32	6/32	4/32	6/32	72	34	53	Bright and clear, brisk breeze
9	5/32	6/32	4/32	6/32	82	42	62	Cloudy, light breeze
10	....	....	....	†	83	50	67	Bright and clear, brisk wind
11	....	....	....	†	70	45	57	Bright and clear, high wind
12	22/32	22/32	20/32	†	68	40	52	Cloudy, brisk wind
13	....	....	....	†	67	33	50	Bright and clear, brisk wind
14	12/32	14/32	12/32	†	69	37	53	Bright and clear, light wind
15	....	....	....	†	80	40	60	Bright and clear, moderate wind, trace rain
16	....	....	....	†	79	51	65	Bright and clear, brisk wind
17	18/32	14/32	16/32	†	82	48	65	Bright and clear, brisk wind
18	....	....	....	†	75	52	66	Cloudy, very high wind, 1 mm. rain
19	....	....	....	†	44	39	41	Cloudy, high wind, 2.7 cm. rain
20	....	....	....	†	52	34	43	Cloudy, light wind, trace rain
21	26/32	24/32	22/32	†	67	40	54	Cloudy, brisk breeze
22	....	....	....	†	65	42	54	Bright and clear, moderate wind
23	6/32	6/32	6/32	†	79	43	61	Bright and clear, calm
24	....	....	....	†	77	45	61	Bright and clear, brisk wind
25	10/32	8/32	8/32	†	76	47	61	Bright and clear, moderate wind
26	....	....	....	†	81	48	65	Bright and clear, moderate breeze
27	12/32	10/32	10/32	†	84	55	69	Bright and clear, moderate breeze
28	....	....	†	†	80	50	65	Bright and clear, high wind
29	....	....	†	†	83	55	69	Cloudy, high wind
30	....	....	†	†	83	52	68	Bright and clear, moderate breeze
31	44/32	44/32	†	†	80	48	64	....
Total or average.	8.28	6.41	8.60	0.38	74	48	58	....

* Pan leaking. † No record available. ‡ Pan submerged.

Date, 1914, April.	Evaporation, in Inches. Land-pans. No. 1.	No. 8	Floating-pans. No. 2.	No. 4.	Temperatures. Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.	Weather conditions.
1	12/32	10/32	‡	†	99	52	71	Bright and clear, perceptible wind
2	....	....	‡	†	86	55	70	Bright and clear, perceptible wind
3	10/32	8/32	‡	†	86	52	69	Bright and clear, moderate breeze
4	....	....	....	†	79	52	66	Bright and clear, moderate breeze
5	....	....	....	†	86	54	69	Bright and clear, high wind
6	14/32	16/32	10/32	†	90	61	76	Cloudy, high wind
7	....	....	....	†	87	57	72	Bright and clear, high wind
8	38/32	36/32	30/32	†	82	55	68	Bright and clear, high wind
9	....	....	....	†	75	42	59	Bright and clear, moderate breeze
10	....	....	....	†	83	59	71	Bright and clear, high wind
11	....	....	....	†	89	57	73	Bright and clear, high wind
12	....	....	....	†	81	57	69	Bright and clear, calm
13	....	....	....	†	81	50	66	Bright and clear, moderate breeze
14	64/32	60/32	66/32	†	88	48	68	Bright and clear, calm
15	....	....	....	†	95	54	74	Bright and clear, perceptible breeze
16	28/32	24/32	14/32	†	94	61	79	Bright and clear, high wind
17	....	....	....	†	92	64	78	Cloudy, high wind
18	30/32	26/32	28/32	†	88	53	70	Hazy, moderate breeze
19	....	....	....	†	88	50	69	Bright and clear, moderate breeze
20	22/32	20/32	20/32	†	93	50	71	Cloudy, moderate breeze
21	....	....	....	†	90	62	76	Bright, clear A. M.; cloudy P. M.; high wind
22	....	....	....	†	93	63	78	Bright, clear A. M.; cloudy P. M.; trace rain
23	....	....	....	†	95	59	77	Bright and clear, high wind
24	....	....	....	†	91	50	70	Bright and clear, high wind
25	44/32	26/32	42/32	†	91	63	77	Cloudy, high wind
26	....	....	....	†	98	57	78	Hazy, moderate breeze
27	32/32	32/32	32/32	†	98	57	77	Hazy, calm
28	....	....	....	†	90	57	74	Cloudy, calm
29	24/32	32/32	....	†	99	54	76	Hazy, calm
30	14/32	16/32	17/32	†	98	54	76	Clear, perceptible breeze
Total or average	10.38	9.62	8.79	†	89	55	72	....

† No record available. ‡ Pan submerged.

Date, 1914, May.	Evaporation, in Inches. Land-pans. No. 1.	No. 8.	Floating-pans. No. 2.	No. 4.	Temperatures. Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.	Weather conditions.
1	....	....	....	†	93	59	75	Clear, perceptible breeze
2	....	....	....	†	98	59	79	Cloudy, calm
3	42/32	48/32	47/32	†	95	57	76	Bright and clear, high wind
4	....	....	....	†	96	52	74	Bright and clear, high wind
5	....	....	....	†	93	52	72	Bright and clear, moderate breeze
6	40/32	48/32	38/32	†	98	57	77	Bright and clear, light wind
7	....	....	....	†	98	59	79	Bright and clear, calm
8	31/32	32/32	30/32	†	96	61	79	Bright and clear, moderate breeze
9	....	†	....	†	100	66	83	Bright and clear, moderate breeze
10	34/32	†	30/32	†	104	66	84	Bright and clear, calm
11	16/32	10/32	12/32	†	100	66	83	Hazy A. M., bright P. M.; light breeze
12	....	....	....	†	99	71	85	....
13	30/32	32/32	28/32	†	94	66	80	Cloudy, moderate wind, trace rain
14	....	....	....	†	83	59	71	Bright and clear, moderate wind, 8.5 mm. rain
15	31/32	17/32	15/32	†	95	53	74	Cloudy A. M., bright and clear P. M.
16	18/32	12/32	28/32	†	95	61	78	Bright and clear, moderate breeze
17	....	....	....	†	100	64	82	Bright and clear, perceptible breeze
18	....	....	....	†	97	63	80	Clear, light breeze
19	....	....	....	†	95	68	82	....
20	....	....	....	†	92	57	74	....
21	....	....	....	†	94	63	79	....
22	....	....	....	†	94	63	78	....
23	108/32	41/32	112/32	†	95	68	82	Bright and clear, light wind
24	....	....	....	†	100	65	82	Bright and clear, cloudy toward evening
25	....	....	....	†	94	64	79	Clear day, light wind
26	24/32	24/32	30/32	†	96	61	79	No. 1 gauge, 3/16 in. rainfall
27	....	....	....	†	98	64	81	....
28	....	....	....	†	100	63	81	....
29	....	....	....	†	98	70	84	....
30	50/32	40/32	32/32	†	97	62	80	....
31	13/32	†	†	†	88	62	75	....
Total or average.	13.75	9.60	12.66	†	96	62	79	....

† No record available.

Date, 1914, June.	Evaporation, in Inches. Land-pans. No. 1.	No. 8.	Floating-pans. No. 2.	No. 4.	Temperatures. Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.	Weather conditions.
1	....	†	†	†	95	61	78	
2	....	†	†	†	95	61	78	
3	39/32	†	†	†	101	84	85	No. 1 gauge shows 5/8 in. rainfall.
4	....	....	....	†	99	69	84	
5	....	....	....	†	99	71	85	
6	....	33/32	32/32	†	101	71	85	
7	....	....	....	....	99	66	82	
8	....	....	....	....	100	68	84	
9	....	....	....	....	101	68	85	
10	....	....	....	....	99	64	81	
11	....	....	....	....	98	64	81	
12	....	....	....	....	98	63	81	
13	....	....	....	....	99	68	84	
14	....	....	....	....	100	70	85	
15	....	....	....	....	96	66	81	
16	....	....	....	....	87	66	76	
17	....	....	....	....	89	63	76	
18	....	....	....	....	90	64	77	
19	....	....	....	....	95	66	80	
20	....	....	....	....	93	66	80	
21	....	....	....	....	94	66	80	
22	....	....	....	....	88	64	76	
23	....	....	....	....	85	65	77	
24	....	....	....	....	90	66	78	
25	....	....	....	....	93	66	80	
26	....	....	....	....	96	68	82	
27	....	....	....	....	99	70	85	
28	....	....	....	....	100	66	83	
29	....	....	....	....	101	68	84	
30	....	....	....	....	101	68	85	
Total or average.	1.22	1.03	1.00	†	96	66.5	81	

† No record available.

NOTES

This table is a record of evaporation and temperature observations, taken after the compilation of Table 34; and is a continuation of that table, to which reference is made for descriptions and locations of evaporation pans and other data.

The general scheme of Table 34 is followed in this table, except that the El Paso records, considered unsatisfactory, were discontinued in March, 1914; and that observations, here given, covering periods of several days' evaporation, have not been distributed over those days but recorded opposite the date on which the reading was made.

Evaporation records for March 1st to 9th, 1914, taken from Table 34, are given as on that table, i. e., distributed over the data covered, assuming that the daily rate of evaporation for each period was constant.

Evaporation given is net, i. e., rainfall has been accounted for.

evaporation depth (the winter months, as March), the month's evaporation depth at any stated mean monthly temperature may vary greatly in different years. For example, at a mean monthly temperature of 50° Fahr., the monthly evaporation depth at El Paso averages 3.7 in. (see the "straight-line" curve of Fig. 7); but for the different months or years observed, the month's evaporation depth at 50° varies from 2.0 to 4.8 in., or from 54 to 130% of the mean value, and from 100 to 240% of the minimum value.

Also, each of the points plotted on Fig. 7 is for a period of a full month, with all the equalizing of the changing daily evaporation conditions which thus would be effected; while the observations at El Paso in March, 1914, at approximately 51° mean temperature, covered a period of only 16 days, and thus are likely to contain more vagaries and uncertainties and to be less equalized than values derived from observations covering a full month or longer.

Finally, the evaporation depth at El Paso for the full month of March, 1914, averages (as deduced from the 16 days' observations) 2.48 in., or is 33% below the average monthly depth at El Paso at 50°; but it could not be known whether the observed evaporation depth at Lake Conchos for the same month of March varied from its mean in the same or in the opposite direction.

For the reasons given, it was concluded that estimates of yearly evaporation depth based on comparisons of evaporation depths at two widely separated stations observed for short periods only, are unsafe and unreliable, and not worthy of consideration.

Variants B_a and B_b consisted of an expansion (in March, 1914) of the measured pan evaporations at Lake Conchos for the months of January-March (the observations covered only 2 months actual time) into the probable depth of pan evaporation for a complete yearly cycle; and later (in August, 1914), of a similar expansion of the longer-continued pan evaporations of January-June (only 5 months actual time).

The B_a method of such expansion is the division of the observed evaporation depth in pans at Lake Conchos of the 3 months, January-March, by the proportion of the evaporation depth for a full year which is believed to occur during those 3 months (as shown by the evaporation-temperature study of Table 31)—and similarly for the longer-continued pan evaporations of the 6 months, January-June, 1914.

The observed evaporation depths at Lake Conchos from land and floating-pans are summarized in Table 35 for the shorter period (January 13th to March 9th, 1914); and for the longer period (January 13th to June 6th, 1914), they are summarized (from Tables 34 and 36, Plates XXXV and XXXVI) in Table 39.

From Table 35, the measured evaporation depths from 3-ft. square floating-pans at Lake Conchos during the shorter period were as given in Table 38, after expanding all the observations of partial months to full months and taking 80% of the evaporation depths from the land-pans as the corresponding depths from floating-pans.

TABLE 38.—Evaporations from 3-Ft. Square Floating-Pans.

Month.	Pan No. 1 (land-pan). Inches.	Pan No. 2 (floating-pan). Inches.	Pan No. 3 (land-pan). Inches.	Pan No. 4 (floating-pan). Inches.	Average of the four pans Inches.
January, 1914..........	*6.36	4.96	*5.57	6.82	5.92
February, "	4.25	5.60	3.57	2.94	4.09
March, "	4.40	5.12	5.44	5.74	5.17
Totals and average....	15.01	15.68	14.58	15.50	{ 15.18 15.19
Percentage............	99.0	103.2	96.0	102.1	100

* The evaporation depths given in this table for Pans Nos. 1 and 3 (land-pans) are 80% of the evaporations actually measured in those land-pans.

From Table 31, the proportions of the total yearly evaporation depths at Lake Conchos which occur in the 3 months, January, February, and March, are as follows:

Depth in January.......... 4.8% of total depth in year.
" " February .... 5.6 " " " " "
" " March .... 7.4 " " " " "
Depth for the 3 months.....17.8% " " " " "

Hence, if 15.18 in. is 17.8% of the yearly evaporation depth, the total yearly depth must be (15.18 ÷ 0.178) = 85.3 in.; or the total yearly evaporation depth at Lake Conchos from 3-ft. square floating-pans is (by Method B_a and for the shorter period) 85.3 in.

By Subsidiary Conclusion (*c*), the evaporation depth from a large reservoir is 62% of that from a 3-ft. square pan floating thereon; hence, by Method B_a and for the shorter period, the yearly evaporation depth from Lake Conchos is (85.3 × 0.62) = 52.9 in.

The yearly evaporation depth by Method B_a for the longer period is obtained from Table 39.

From Table 39, the measured evaporation depths from the four 3-ft. square pans were as given in Table 40 for the longer period, after expanding all observations for parts of months to full months and modifying the depths measured in the land-pans to their corresponding depths (80%) in floating-pans.

TABLE 39.—SUMMARY OF PAN-EVAPORATION MEASUREMENTS MADE AT LA BOQUILLA (LAKE CONCHOS) IN 1914.

Evaporation Pan.	Pan No. 1 (Land-Pan).						Pan No. 2 (Floating-Pan).					
Month (of 1914).	Jan.	Feb.	Mar.	Apr.	May.	June.	Jan.	Feb.	Mar.	Apr.	May.	June.
Number of days measured	19	27	31	30	31	3	9	21	22	27	30	3
Total depth measured, in inches	4.87	5.1	6.28	10.38	13.75	1.22	1.44	4.2	3.60	8.72	12.56	1.00
Average daily evaporation depth, in inches	0.26	0.19	0.20	0.35	0.44	0.41	0.16	0.20	0.16	0.32	0.42	0.33
Corresponding depth for full month, in inches	7.95	5.30	6.28	10.38	13.75	12.20	4.96	5.60	5.07	9.70	12.98	10.00
Depth for 3-ft. floating-pan, for month, in inches	6.36	4.25	5.02	8.30	11.00	9.76	4.96	5.60	5.07	9.70	12.98	10.00
Mean temperature, in degrees, Fahrenheit, for period of measurement.	64	56	58	72	79	80	64.5	56	58	73	79	85
Mean temperature, in degrees, Fahrenheit, for entire month	62	56	58	72	79	81	62	56	58	72	79	81

Evaporation Pan.	Pan No. 3 (Land-Pan).						Pan No. 4 (Floating-Pan).					
Month (of 1914).	Jan.	Feb.	Mar.	Apr.	May.	June.	Jan.	Feb.	Mar.	Apr.	May.	June.
Number of days measured	18	22	31	30	28	3	2	6	2	......	......	......
Total depth measured, in inches	4.04	3.5	6.41	9.63	9.59	1.03	0.44	0.63	0.37	......	......	......
Average daily evaporation depth, in inches	0.22	0.16	0.21	0.32	0.34	0.34	0.22	0.10	0.19	......	......	......
Corresponding depth for full month, in inches	6.96	4.46	6.41	9.63	10.62	10.30	6.82	2.94	5.74	......	......	......
Depth for 3-ft. floating-pan, for month, in inches	5.57	3.57	5.13	7.70	8.50	8.24	6.82	2.94	5.74	......	......	......
Mean temperature, in degrees, Fahrenheit, for period of measurement.	64	56	58	72	79	85	60	58	57	......	......	......
Mean temperature, in degrees, Fahrenheit, for entire month	62	56	58	72	79	81	62	56	58	......	......	......

TABLE 40.—EVAPORATIONS FROM 3-FT. SQUARE FLOATING-PANS.

Month.	Pan No. 1 (land-pan). Inches.	Pan No. 2 (floating-pan). Inches.	Pan No. 3 (land-pan). Inches.	Pan No. 4 (floating-pan). Inches.	Average of the four pans. Inches.
January	6.36	4.96	5.57	6.82	5.92
February	4.25	5.60	3.57	2.94	4.09
March	5.02	5.07	5.13	5.74	5.24
April	8.30	9.70	7.70	*....	8.57
May	11.00	12.98	8.50	....	10.82
June	9.76	10.00	8.24	....	9.33
Totals and averages	44.69	48.31	38.71	....	{ 43.97 43.90
Percentages	101.9	110.0	88.2	....	100

* Observations in Pan No. 4 were abandoned March 9th, because of too frequent submergences of this pan by waves.

From Table 31, the proportions of the total yearly evaporation depth at Lake Conchos occurring in each of the 6 months, January-June, are as follows:

Depth in January....... 4.8% of total depth in year.
" " February ..... 5.6% " " " " "
" " March .... 7.4% " " " " "
" " April .... 9.4% " " " " "
" " May ....11.5% " " " " "
" " June ....11.7% " " " " "
Total for the 6 months. .50.4% of total depth in year.

Hence, on the assumption that 50.4% of the yearly evaporation depth equals 43.97 in., the total yearly evaporation depth at Lake Conchos must be $(43.97 \div 0.504) = 87.2$ in. from 3-ft. square floating-pans and—by Method B_a and for the longer period—$(87.2 \times 0.62) = 54.1$ in. from the surface of the lake itself.

The expansion by Method B_b from the 3 and 6 months' observed evaporations to that for a full year is made by evaporation-temperature relations, as follows:

By Method A, the yearly evaporation depth from 3-ft. square pans floating on Lake Conchos is 84.74 in. (see Table 31); and (also from Table 31) the evaporation depths for the 3 months, January-March, and the 6 months, January-June, should be, respectively $(4.09 + 4.75 + 6.23) = 15.07$ and $(4.09 + 4.75 + 6.23 + 7.95 + 9.72 + 9.90) = 42.64$ in.

From Tables 38 and 40, the average pan evaporations (from 3-ft. square floating-pans) actually observed at Lake Conchos were as follows:

3 months, January-March.................15.18 in.
6 " January-June..................43.97 "

From Tables 35 and 39 and Fig. 15, the observed mean monthly temperatures at Lake Conchos and their corresponding theoretical monthly evaporation depths were as given in Table 41.

TABLE 41.—THEORETICAL EVAPORATION DEPTHS FROM 3-FT. SQUARE FLOATING-PANS, CORRESPONDING TO OBSERVED MEAN MONTHLY TEMPERATURES.

Month.	January-March, 1914.		January-June, 1914.	
	Observed mean monthly temperature, in degrees, Fahrenheit.	Corresponding theoretical monthly evaporation depth, in inches.	Observed mean monthly temperature, in degrees, Fahrenheit.	Corresponding theoretical monthly evaporation depth, in inches.
January.......	62	6.03	62	6.03
February.....	56	4.76	56	4.76
March........	55	4.55	58	5.20
April..........	..	....	72	8.16
May...........	..	....	79	9.64
June..........	..	....	81	10.05
Totals.....	....	15.34	....	43.83

Hence the comparison of the observed evaporation depths at Lake Conchos with the theoretical depths deduced by evaporation-temperature relations are as given in Table 42.

TABLE 42.—COMPARISON OF OBSERVED AND THEORETICAL EVAPORATION DEPTHS.

Item.	3 months—January-March.	6 months—January-June.
Observed evaporation depths....	15.18 in. = 100%	43.97 in. = 100%
Theoretical evaporation depths :		
From temperatures of 1912-13.	15.07 " = 99.3%	42.64 " = 97.1%
" " " 1914....	15.34 " = 101.1%	43.84 " = 99.8%
Mean of two..................	100.2%	98.45%

Thus, from the results of the three (2) months' pan observations at Lake Conchos, the theoretical evaporation depths there (those estimated by Method *A*) apparently are 0.2% too great; and from the results of the six (5) months' observations, 1.55% too small. The evaporation depths at Lake Conchos, as estimated by Method B_b, therefore, are as given in Table 43.

The estimation of the evaporation depth by Method *C* will now be considered.

Method C.—This method consists in estimating the inflow to the lake throughout two practically rainless periods, deducting therefrom

the measured outflow during such periods, and then, from the observed elevations and known areas of the lake surface, estimating the depth on the lake surface corresponding to the excess of inflow over outflow, which excess must have been (at least to a close approximation) the loss by evaporation.

TABLE 43.—YEARLY EVAPORATION DEPTHS AT LAKE CONCHOS, BY METHOD B_b.

From three (2) months' pan observations.	From six (5) months' pan observations.
In 3-ft. square floating-pans: *84.74 (= A) ÷ 1.002 = 84.5 in. = 98.2%	In 3-ft. square floating-pans: 84.74 (= A) ÷ 0.9845 = 86.2 in. = 100%
In Lake Conchos: *52.5 (= A) ÷ 1.002 = 52.4 in. = 98.2%	In Lake Conchos: 52.5 (= A) ÷ 0.9845 = 53.4 in. = 100%

*From Table 33.

The two periods thus used are the 4 months, October, 1913-January, 1914, and the 7 months, October, 1913-April, 1914. Since January, 1912, continuous gaugings have been made of the flow of the Rio Conchos at Pilar de Conchos, 40 miles up stream from the La Boquilla Dam and 2 miles above the upper end of the filled reservoir. There are a number of arroyos between Pilar de Conchos and the dam, however, which discharge much water into the lake during rainy and flood seasons; and it is only during rainless periods that the Rio Conchos discharge, as measured at Pilar de Conchos, constitutes the total water supply to the reservoir. The 7 months, October, 1913-April, 1914, were such a practically rainless period.

The inflow and outflow of Lake Conchos during the 4 and 7-month periods are estimated from the continuous stream gaugings, at Pilar de Conchos and just below the La Boquilla Dam, to have been as given in Table 44.

TABLE 44.—EXCESS OF INFLOW TO LAKE CONCHOS OVER OUTFLOW.

Item.	MILLIONS OF CUBIC METERS.	
	4 months, October-January.	7 months, October-April.
Inflow as gauged at Pilar de Conchos.......... ..	44.193	59.184
Outflow as gauged just below La Boquilla Dam.	*20.095	†36.963
Gross volume of water reservoired...............	24.098	22.221

* These volumes include, not only the stream flows as measured just below the power-house, but also the measured seepage losses entering the stream below that gauging station, plus 10% of such measured seepage losses added to cover known but unmeasurable additional seepage losses.

† Due to fuller and more complete field reports at the later date, the seepage losses of October–January are larger and more exact and complete in the volumes of outflow in Table 44 for October–April than in the volumes for October–January only given in the table separately.

The fluctuations of the lake surface during the two periods are given in Table 45.

TABLE 45.—ELEVATIONS AND RISES OF SURFACE OF LAKE CONCHOS, ETC.

Item.	4 months, October-January.	7 months, October-April.
	Meters.	Meters.
Elevation of surface of lake above sea, at beginning and end of period	1 288.96 and 1 289.28	1 288.96 and 1 288.92
Mean elevation of lake surface during the period	1 289.12	1 288.94
Rise of lake surface during the period	+ 0.320	− 0.04
Rainfall during period (mean of the two rain-gauges at La Boquilla and Pilar de Conchos)	− 0.058	− 0.077
Rise of lake surface after eliminating the effect of rain falling directly on lake; or rise corresponding to net reservoired water [(inflow less outflow), less depth lost by evaporation]	+ 0.262	− 0.117
	Millions of cubic meters.	Millions of cubic meters.
Volume of reservoir storage per meter of rise, at mean surface elevation	*42.4	*42.0
Gross volume of water reservoired during period, including that lost afterward by evaporation, but excluding that gained by direct rainfall (from Table 44)	24.098	22.221
	Meters.	Meters.
Rises of lake surface (at mean elevations of 1 289.12 and 1 288.94 m.) corresponding to above 24.098 and 22.221 millions of cubic meters; or rises which would have occurred if there had been no evaporation loss from lake surface	† + 0.568	† + 0.529
Rises which actually did occur after losses by evaporation and correction on account of rainfall (from above)	+ 0.262	− 0.117
Differences in theoretical and actual rises above; or estimated evaporation depths from the lake in the 4 months, October-January, and the 7 months, October-April [0.568–0.262 m. and 0.529 — (— 0.117) m.]	0.306	0.646
Estimated evaporation depths from the lake during the 4 months, October-January, and the 7 months, October-April, in inches (3.28 × 12 × 0.306 and 3.28 × 12 × 0.646)	12.04 in.	25.40 in.

* From curves and tables of the reservoir areas and capacities.

† (24.098 ÷ 42.4) = 0.568, and (22.221 ÷ 42.0) = 0.529.

During May, 1914, the rainy season began, and many local flood flows into the lake occurred from arroyos between Pilar de Conchos and La Boquilla. Hence the measurement of the inflow to the lake and the use of Method *C* were impracticable after the end of April, 1914.

The estimated evaporations, of 12.04 in. in the 14 months and 25.40 in. in the 7 months, were expanded into the evaporation depths for a full year by two variants, C_a and C_b.

Method C_a is identical with that used in Method B_a, and consists in expanding by making use of the proportion of the total yearly evaporation depth which occurs in the 4 and 7 months, as in Table 46 (from Table 31).

TABLE 46.—THE PROPORTIONS OF THE TOTAL YEARLY EVAPORATION DEPTH OCCURRING DURING THE 4 MONTHS, OCTOBER-JANUARY, AND THE 7 MONTHS, OCTOBER-APRIL.

Month.	4 months, October-January.	7 months, October-April.
October	7.7%	7.7%
November	5.1	5.1
December	4.1	4.1
January	4.8	4.8
February	...	5.6
March	...	7.4
April	...	9.4
Totals	21.7%	44.1%

Hence, if 12.04 in. = 21.7%, and 25.40 in. = 44.1%, of the total yearly evaporation depth from the surface of Lake Conchos, the total yearly evaporation depth, as estimated by Method C_a, must be:

From the consideration of the 4 months,
October-January $(12.04 \div 0.217) = 55.4$ in.
From the consideration of the 7 months,
October-April $(25.40 \div 0.441) = 57.7$ in.

Method of expansion C_b is identical with that used in B_b, and consists in expanding from the 4 and 7 months to the full year by making use of evaporation-temperature relations, as follows:

By Method A, the yearly evaporation depth from the surface of Lake Conchos is 52.5 in. (see Table 33).

From Tables 37 and 39, and Fig. 15, the observed mean monthly temperatures at Lake Conchos and their corresponding theoretical monthly evaporation depths were as given in Table 47.

By the relation adopted, 62%, the theoretical evaporation depths in Table 47 from 3-ft. square floating-pans would correspond to the following evaporation depths from the surface of Lake Conchos:

From the consideration of the 4 months,
October-January $(18.18 \times 0.62) = 11.25$ in.
From the consideration of the 7 months,
October-April $(36.30 \times 0.62) = 22.50$ in.

TABLE 47.—THEORETICAL EVAPORATION DEPTHS FROM 3-FT. SQUARE FLOATING-PANS, CORRESPONDING TO OBSERVED MEAN MONTHLY TEMPERATURES.

Month.	4 MONTHS, OCTOBER-JANUARY.		7 MONTHS, OCTOBER-APRIL.	
	Observed mean monthly temperature, in degrees, Fahrenheit.	Corresponding theoretical monthly evaporation, in inches.	Observed mean monthly temperature, in degrees, Fahrenheit.	Corresponding theoretical monthly evaporation, in inches.
October.....	60	5.60	60	5.60
November..	52	3.90	52	3.90
December...	46	2.65	46	2.65
January....	62	6.03	62	6.03
February...	..	....	56	4.76
March.......	..	....	58	5.20
April........	..	....	72	8.16
Total......	..	18.18	..	36.30

The above theoretical evaporations from the surface of the lake, estimated by Method A, are, respectively (11.25 ÷ 12.04*) = 93.5% and (22.50 ÷ 25.40*) = 88.6% of those estimated by Method C; and, assuming the same relations to hold for the entire year and to apply to the 52.5 in. yearly evaporation depth estimated by Method A, the estimated yearly evaporation depth from the surface of the lake is as follows by Method C_b:

From the consideration of the 4 months,
October-January (52.5 ÷ 0.935) = 56.2 in.
From the consideration of the 7 months,
October-April (52.5 ÷ 0.886) = 59.3 in.

For various reasons mentioned on pages 1735 to 1738, it is believed that nearly all the uncertainties incident to the estimation of the yearly evaporation depth from Lake Conchos by Method C are of a nature tending to make the depths resulting from that method somewhat greater than the actual; or that the true yearly evaporation depth in all probability is somewhat less than the depths of 55.4 and 57.7 in. and 56.2 and 59.3 in. resulting from Methods C_a and C_b, respectively.

In Tables 3 and 4 are given a summary and a percentage comparison of the five values of yearly evaporation depth from Lake Conchos estimated (for the longer period of observation) by methods A, B_a, B_b, C_a, and C_b. In Table 5 is given a percentage-comparison of the five values, as estimated from both the longer and the shorter periods. On page 1738 is given the value of 55 in. (1.40 m.) adopted in March,

* From Table 45.

1914 (and confirmed in August), as the safe gross yearly evaporation depth from the surface of the lake, and the reasons for that adoption. And in Table 6 is given the division of the 55 in. yearly depth into the depths of the 12 months, also the net monthly evaporation depths after reducing the gross depths by the average depth of rain each month falling directly on the reservoir surface.

This completes the Appendix and this study of evaporation—except that data now are available to check, from the Lake Conchos investigation itself, the value of 62% which was adopted as Subsidiary Conclusion (*c*).

(*c*). Evaporation Depth from a Large Reservoir, as a Percentage of That from a 3-ft. Square Pan Floating Thereon. (As Checked from Lake Conchos Data.)

The yearly evaporation depth from Lake Conchos itself has been estimated from 4 months' and 7 months' observations by Methods C_a and C_b. The yearly evaporation depth from 3-ft. square pans floating on the lake have been estimated by Method A, and from 2 months' and 5 months' observations by Methods B_a and B_b. All these yearly depths of evaporation are summarized in Tables 48 and 49.

TABLE 48.—YEARLY DEPTH OF EVAPORATION FROM THE SURFACE OF LAKE CONCHOS.

Method of estimation.	From Lake Observations of:	
	4 months.	7 months.
C_a — from page 1788...........	55.4 in. = 99.3%	57.7 in. = 98.8%
C_b — from page 1789...........	56.2 " = 100.7%	59.3 " = 101.2%
Mean of C_a and C_b	55.8 in. = 100%	58.5 in. = 100%

TABLE 49.—YEARLY DEPTH OF EVAPORATION FROM 3-FT. SQUARE PANS FLOATING ON LAKE CONCHOS.

Method of estimation.	From Pan Observations of:	
	2 months.	5 months.
*A—from Table 32.............	*84.7 in. = 99.8%	*84.7 in. = 97.7%
B_a— " pages 1782-1784.....	85.3 " = 100.5%	87.2 " = 100.6%
B_b— " Table 43............	84.5 " = 99.5%	86.2 " = 99.4%
Mean of A, B_a, and B_b.........	84.85 in. = 100%	86.03 in. = 99.2%
Mean of B_a and B_b	84.9 " = 100%	86.7 " = 100%

* Method A includes no pan observations at Lake Conchos, but temperature observations only.

Various combinations of Tables 48 and 49, giving various values of c, are given in Table 50:

TABLE 50.—Various Values of c from Lake Conchos Data.

Combinations of lake evaporation and pan evaporation.	From Observations of:	
	4 and 2 months.	7 and 5 months.
$C_a \div B_a$	$(55.4 \div 85.3) = 65.0\%$	$(57.7 \div 87.2) = 66.2\%$
$C_b \div B_b$	$(56.2 \div 84.5) = 66.5$	$(59.3 \div 86.2) = 68.8$
*Mean of C_a and C_b $\div$ mean of B_a and B_b	$(55.8 \div 84.9) = 65.8$	$(58.5 \div 86.7) =$ *67.5
Mean of C_a and C_b $\div$ A	$(55.8 \div 84.7) = 65.9$	$(58.5 \div 84.7) = 69.1$
Mean of C_a and C_b $\div$ mean of A, B_a, and B_b	$(55.8 \div 84.85) = 65.8$	$(58.5 \div 86.03) = 68.0$
Mean of all above values	65.8%	67.9%

* Believed to be the most accurate and reliable value.

The values of c in Table 50 vary through only a small range, the difference between the highest and lowest values (69.1 and 65.0%) being only 4.1 per cent.

The 67.5% is felt to be the most accurate and reliable value of all —because of the longer periods of observation, because the lake and the pan observations of Methods C and B cover nearly the same periods of time, and because Method B makes use of the local pan evaporations at Lake Conchos while Method A does not.

The 67.5% value of c corresponds to yearly evaporations of 58.5 in. from the surface of the lake and 86.7 in. from 3-ft. square pans floating thereon. As has been stated already, it is known that the evaporation depths estimated by Method C err in being too great, and that, to secure accurate values, they must be decreased by the depths of water absorbed by the bottom of the reservoir during the period of observation (which was its first time of filling).

The present operation is a check on the correctness of the value 62% for c. Assuming that 62% be the correct value, then the yearly evaporation depth from the surface of Lake Conchos would be (86.7 in. pan evaporation $\times$ 0.62) $=$ 53.75 in. This is only (58.5 — 53.75) $=$ 4.75 in. less depth than the most reliable value of lake evaporation as estimated by Method C. It is shown in the discussion of Table 4, however, that there is no improbability and even much probability of the absorption, etc., even of 5.05 in. depth of the reservoired water during the period of observation.

Hence, from the check of the value of c by the Lake Conchos observations, it may be stated that those observations show the evaporation depth from large reservoirs to be certainly less than 67.5% (and in all probability as little as 62%) of that from 3-ft. square pans floating thereon.

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PAPERS AND DISCUSSIONS

THE VALUATION OF PUBLIC UTILITY PROPERTY

Discussion.*

By J. H. Gandolfo, Assoc. M. Am. Soc. C. E.†

J. H. Gandolfo,‡ Assoc. M. Am. Soc. C. E. (by letter).—In replying to the discussion of his paper, it is the writer's intention to treat the subject first in a general way, and then take up each contribution separately, making such comments on the salient points therein as seem necessary in each individual case which has not been covered in the general discussion. Mr. Gandolfo.

Few people, and much less engineers, particularly those who have discussed this paper, seem to realize the changed conditions surrounding public utilities, and public utility properties, that exist to-day, compared with what they were in the past; and it is not necessary to state that such changing conditions are probably destined to continue to a greater or less degree. Only a few years ago the regulation of a public utility by the State, in the smallest degree, was considered an attack on individual liberty and the right to hold property. As for an appraisal by the State, and the regulation of rates and security issues, such things were looked on as preposterous, and nothing short of revolutionary. Now all is changed, and we find the public utility, as a general rule, accepting all these things with equanimity. The public utility has become almost if not entirely the agent of the State, and as such is answerable in detail to the State.

From the writer's knowledge of the conception and development of private enterprises, and his connection with public utility developments, he fails to see any valid reason for not conducting and de-

* Discussion of the paper by J. H. Gandolfo, Assoc. M. Am. Soc. C. E., continued from February, 1915, *Proceedings*.

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Mr. Gandolfo. veloping a corporation, whether a private or public utility, according to the same general principles of honest business endeavor as a conservative and carefully managed private business. There is no reason for having one set of morals or code of ethics for the private business man, and an entirely different set for the corporation and the men who are responsible for its being and management.

The following is an example of what has been considered perfectly legitimate financing in corporate development. An appraisal of an industrial plant with which the writer was connected, and an investigation of the general business and financial conditions surrounding it, disclosed the following condition:

Actual cost of plant, including all buildings and machinery................	$1 100 000
Appreciated value of the land (the actual cost some years before was $45 000)..	250 000
Working capital......................	500 000
Total valuation................	$1 850 000

The capitalization of this plant was:

Bonds............................	$3 000 000
Preferred stock......................	1 500 000
Common stock......................	1 500 000
Total capitalization..........	$6 000 000

(In this particular investigation, the present value of the plant would have had no possible significance.)

What would be thought of the private individual who, for every dollar that he wished to invest in a plant, issued obligations to the amount of nearly $3.25, obligations that constitute a lien against his plant, and that must be met and settled some day?

If a business man invests $1 000 000 in a private business, that is the value of his plant. If he wishes to raise money on it in any way, that is the figure which would be used, provided depreciation and obsolescence had not taken place. In the past, however, it has been the general idea that a corporation, whether a private or public utility, could issue securities of two, three, or four times the amount of the actual investment. Such securities, whether legally or not, are at least morally and ethically a "mortgage" on the plant. It is on these securities that the public must pay interest. It is such securities that constitute a burden on future generations, for ultimately payment or default is a certainty. This over-capitalization is one reason for so many public utility corporations being so insistent on the use of the present-value theory of appraisal, generally without any

allowance for depreciation, because, on account of the generally increasing cost to reproduce such plants (as the writer has already pointed out), this theory gives a larger tangible value behind such securities. Mr. Gandolfo.

The argument is often advanced that a corporation has no soul, that it is simply a great machine, grinding along irrespective of any one or any thing. Only a few months ago the manager of a large engineering corporation, which has built and controls many public utilities of all kinds, advanced this very argument to the writer. But, stop a moment, and briefly analyze the corporation. Men are responsible for its very existence. Men are responsible for all its acts and all its results. In other words, a corporation without the human element is a dead and useless thing. It is only the "bare bones" of the organization, without muscle, sinew, or brain. The human element gives it life, makes it a "going concern", and therefore men are as responsible for its acts and there is as much responsibility, morally and financially, resting on those who direct its smallest acts, as if these same people were engaged in private business.

Those who contend that a physical valuation for the purposes of rate-making or security issues should be founded on present value, with or without depreciation, or on replacement value, lose sight of the origin and *raison d'être* of both security issues and rates. They also fail to have analyzed the subject correctly, or to have investigated the anomalies and injustices that arise when any one of these systems is followed. In order to show this, the advocates of such systems are asked two questions:

(1) What are security issues for?
(2) What is a rate of return, as fixed by a commission or other controlling power, for?

Taking up the first question: The issue of legitimate securities is for the purpose of obtaining, by their sale, funds to invest in some enterprise, or to cover the value of money and labor already invested. Any other issue of securities, not backed up by good and sufficient assets (and included in assets should be such items as promotion fees, organization expenses, etc.), amounts to adding "water" to an otherwise legitimate enterprise.

Answering the second question: A rate of return is supposed to be such an income as will give a fair percentage of return on the capital invested in the enterprise. Such rate of return, of course, should be higher than the prevailing rate of interest, otherwise there would be no "profit" in the undertaking. The present or replacement value does not give the capital invested in an enterprise, and therefore a rate of return founded on such a value does not give a return on the capital invested.

Mr. Gandolfo. To show how the present or replacement value, as a basis for the issue of securities, or as a basis of rate-making, may work great injustice to innocent investors, or to the rate-paying public, will be illustrated by the following example: The cost of the steelwork for three large power-houses, with the design and construction of which the writer is thoroughly familiar, is used as an illustration, as here is a standard which is probably affected less by unforeseen conditions and other contingencies than any other. For example, foundations differ widely in cost, due to sub-surface conditions, superstructions due to architectural treatment, etc. The construction of all three of these power-houses was similar, the main features being boiler-rooms and engine-rooms separated by a division wall, over-head coal bunkers supported on the steel columns, conveyor runways, monitors, etc.

These three power-houses will be designated as A, B, and C, and the year of erection of the steelwork, and the costs, were as follows:

Power-house.	Year erected.	Cost of steel per pound, erected.
A	1902	3.74 cents.
B	1906	4.07 "
C	1909	2.87 "

Now suppose that a physical valuation of the steelwork for A was made at the time B was built, and that present or replacement value was used, and, in order not to complicate the problem unnecessarily, that no allowance for depreciation was made. This valuation would raise the value of the steelwork for A $8\frac{1}{3}$ per cent. If additional securities were to be issued to cover this increased value of the plant, the first question that would naturally arise is: To whom should they be issued? If they are to be issued at all, they must be issued *pro rata* to the security owners of record on the day such new securities were issued.

Now comes the ethical and practical question as to what is to be done with the rate of return for A, assuming that the rate has already been fixed by law, on the basis of the original value, and that perhaps such rate has been tested and finally decided by the Courts. Should it be suddenly raised to pay a "fair return" on this new valuation, or should it be left at the old rate, which in percentage would now show a less than "fair return"? The utility has made no increase or betterment in service. It has invested no new capital. It has done nothing to benefit or help the rate-payer. This increase in value amounts to an unearned increment due to extraneous conditions over which no one has any control. If the utility is really the agent of the State, should not this increase in value belong to the principal, not to the agent?

Now assume that a physical valuation of the steelwork for both power-houses, A and B, was made at the time C was built, and that present value was used. Under these conditions the value of the steel work for A would decrease more than 23%, and that for B more than 29 per cent. Following out the same line of reasoning, it now becomes necessary to reduce the capitalization of both A and B, in order to conform to the present value as exemplified in C; but who is to give up these securities? The stockholders of record of A and B in 1909 may be and probably are an entirely different set of people from those of 1906 and 1902. Those who paid 3.74 cents in 1902, 4.07 cents in 1906, and those who further received the bonus on A in 1906, may have absolutely no interest in A, B, or C in 1909. To go to innocent parties, and say that they must give up arbitrarily part of their investment simply on account of a falling market for the materials of construction, amounts to the worst form of confiscation of property; and yet this is what the advocates of the present-value, or replacement cost, or replacement-cost-less-depreciation theories of physical valuation, ask us to do, if the cost of the utility happens to have decreased. Mr. Gandolfo.

Then here again, as before described, comes in the question of rate of return. Should rates be immediately reduced, to give a fair return on what is now the "fair value" of the plants? Would such a procedure be fair or just to the utility, any more than the former case would be fair to the rate-payers?

One of the principal objections to the actual-cost method of appraisal is that it deprives some one of the natural increase in value of the utility. In addition to those reasons given in the paper against the capitalization of such increase, is the following:

It is now claimed by many, and seems to be becoming more and more of an accepted fact, that the public utility is in reality only the agent of the State, and acts as such. If this is the case, then assuredly such increase in value belongs to the principal, not to the agent, except such percentage as might be agreed on as reverting to the agent as a part of his remuneration.

Again, who is the actual owner of a public utility? In many cases the records of the New York Stock Exchange show that the entire capital stock of some corporations has changed hands within a comparatively few days. Bonds and gilt-edged investment securities are also constantly changing hands. Unless there is some stable figure on which to base the value of these, investments will be in a constant state of uncertainty. The figure founded on present value is unstable, and is constantly changing.

The writer has also heard the argument advanced that, in a large utility, although there are many elements that have increased in value since it was built, there are also many that have decreased greatly in actual cost, and thus the two sets of elements tend to neutralize

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each other. If this were true, or even only approximately true, there would be no need of further argument and contention between the two schools of appraisal. Each would arrive at the same result by different methods, and all differences of opinion would be at an end.

Several of those discussing the paper have stated in their arguments that if the actual-cost-to-date method of appraisal is used, an expert accountant is all that is necessary to do the work, and that the trained engineer can be entirely eliminated. Such statements are without foundation in fact. An expert accountant is absolutely without the necessary training or experience along engineering lines to make any kind of an appraisal or valuation of a public utility property, no matter what system is used.

To begin with, the expert accountant is without any knowledge as to depreciation in any of its forms. Therefore it is impossible for him to take into account what may be a very important factor, and possibly the most important one, in an appraisal. For example, the writer is at present in a position to take careful note of the condition of the rolling stock of a trolley line. There is not a single car on the entire line in first-class condition. It is a marvel as to how some of them hold together. What would the expert accountant's book investigation show of such a condition? His figures would be the same, whether the plant had 100% in efficiency and condition, or was a scrap heap.

Also, how can the expert accountant know, in many cases, what items should be charged to maintenance, and what to new construction? Or what items on his list of costs have been totally consumed, and have been replaced again and again? Such questions often tax the training and experience of the expert engineer to the utmost, in order to arrive at a just decision.

Secondly, such an investigator would be absolutely incapable of judging whether or not there had been unnecessary extravagance in the design and construction of a public utility property. The writer is familiar with a public utility power-house the superstructure of which was designed by architects totally unfamiliar with this class of work. An expensive flaring base was provided for all walls, entailing also much wider and heavier foundations than would have been necessary otherwise. There was also an expensive overhanging brick cornice, with terra cotta coping, surmounted with a heavy wrought-iron pipe rail. The main entrance vestibule was finished in marble, with marble seats of Roman design along the walls. These are only some of the totally unnecessary architectural features embodied in the design of this power-house which helped to place an unjust burden on the traveling public.

In another public utility power-house, among other totally unnecessary architectural and ornamental features, the turbine-room was finished in expensive tiled brick of different colors, extending from the floor to the crane rail. The crane rail itself was completely hidden by a projecting cornice of the same material. How would an expert accountant be able to judge as to whether or not such features, and others perhaps not so glaringly extravagant, were necessary to the successful and satisfactory, as well as economical, operation of the utility? Mr. Gandolfo.

Finally, how could the expert accountant know whether or not there had been fraud in the entire development and construction of the utility? He could detect straight falsifying of the figures, if this had been resorted to; but such crude methods are not generally used. He would not be able to judge whether unit costs had been unduly increased, whether higher wages had been placed on the books than had actually been paid, whether padded pay-rolls had been certified to; whether unnecessarily high prices had been paid for materials and workmanship; or to detect the thousand and one tricks resorted to by unscrupulous promoters, who care nothing for the ultimate success or failure of an enterprise, but rely solely on the excess cost of construction, or other questionable methods, to reimburse themselves.

By these facts it is easily seen that the expert accountant is totally incapable of making a physical valuation of a public utility property, no matter what theory of valuation is used. Those who advance such an argument show one of two things: Either they are ignorant of many of the necessary requirements of a physical valuation, or they are trying to support an untenable position by resort to arguments which are palpably weak and without foundation.

Several of those who have discussed the paper lay great stress on decisions of the Courts in regard to "value". The term "value" has never been defined. It is simply a matter of the opinion of one man, or of a set of men, and is often influenced by ignorance, prejudice, or bias. It has been said that value is what a thing will bring in the open market. There is no open market for a public utility property, and on analysis, this test also fails in regard to other things. For example, one man will pay, perhaps, $40 000 for a single vase which another man would shatter with a blow of his hammer as being a useless incumbrance. As an example of different opinions as to "value", not only of intangible things, but of concrete physical work, no better case can be cited than that of a recent appraisal of the Chicago elevated railroads. On April 30th, 1912, George F. Swain, Past-President, Am. Soc. C. E., submitted his report on the value of the Chicago elevated railways, giving $34 634 396 as the depreciated value, exclusive of real estate, rights of way, and overhead charges. In this amount most of the uncertain items in a rail-

Mr. Gandolfo. road valuation are omitted. For the same property, A. L. Drum and Company estimated $40 750 892, and the Harbor and Subway Commission of Chicago $26 354 217.

As to judicial decisions, these, too, change with time and circumstances. Furthermore, a Court must decide on the evidence as presented before it. It is the prerogative of the engineer to conclude what is just and right, and then present the matter before the Court so that no alternative is left but to render a decision along just lines. Very often it is not possible to leave this to the attorneys in the case, but the expert engineer must take the lead, in the preparation and presentation of the case, as well as in the mere physical examination and investigation. The writer once heard a late noted lawyer bungle his presentation of a case in Court—in which appraisal and expert opinion played the most important part—so as to astonish all connected with the proceedings. This is what engineers must guard against, in all valuation and appraisal cases with which they have anything to do.

The writer has great respect for the judiciary, and high regard and opinion for the Supreme Court of the United States and its decisions; but all Courts are composed of men, and no man is infallible, and the following instances go to show that even the decisions of this great tribunal may become only matters of history. It is only a short hundred years ago that flogging was practiced in the United States Navy, and was upheld by law. What an outcry would occur if such a thing was attempted now. In 1857 the Supreme Court handed down the famous "Dred Scott Decision". This decision was accepted by not more than half the people of the United States.

As already referred to in the paper, in the decisions relating to the valuation of public utilities, the Courts have several times said that the market value of the securities of a corporation should be given consideration in an appraisal. Does any one who is at all familiar with the fluctuations of securities, and the methods of manipulation as used in the stock markets, suppose that the market value of these necessarily has any possible relation to the physical value of the utility, even though so stated by the Supreme Court of the United States?

As examples of the fluctuations in securities, the following are given. In 1902 the stock of the New York, New Haven, and Hartford Railroad sold at $255 per share. To-day it is selling around $52, a difference of $203 in 12 years. In 1913 it varied from $129⅞ to $65⅝ per share. The stock of the St. Louis and San Francisco Railroad in 1913 sold for $59 per share. In 1914 it sold for $3½ per share. The stock of the New York Central and Hudson River Railroad, one of the leading roads of the country, sold for $174½ in 1902. To-day it is selling for $83. Even such a stock as that of

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the Pennsylvania Railroad sold for $170 per share in 1902, and this year is selling around $105. If space permitted, a long list of both stocks and bonds could be given, with fluctuations even worse than some of those just quoted. This shows the utter fallacy of basing any conclusion on this element. Indeed, it is valueless in an appraisal, even for purposes of comparison.

Mr. Green and Mr. Mayer have hinted that the paper is a socialistic production. Particularly is this so with Mr. Mayer, who refers to the subject again and again in his discussion. The writer wishes to deny emphatically any idea of advancing in any way any socialistic doctrines whatever, or having any leanings whatever toward any of the socialistic propaganda. The paper sets forth some hitherto unpublished arguments and reasons as to why certain theories, already held by many eminent engineers and jurists, are just and reasonable. It is in no sense socialistic.

These two gentlemen have fallen into a mode of procedure that, unfortunately, is quite common to-day, namely, when any one advances arguments which do not happen to coincide with his own, or do not happen to suit certain preconceived notions of some particular section of the community, to try and disparage them, and also their author, by classing them as a socialistic propaganda, particularly if such arguments are otherwise difficult or impossible to refute. Such methods are without justification, and have no place in a scientific discussion before a scientific society.

From the opening paragraphs of Mr. Dow's discussion it is very evident that he does not appreciate the real question at issue; neither does he seem to consider that, in regard to appraisal and valuation, when the question of rates or security issues is at stake, there are two distinct schools of engineers at the present time, one holding that the actual cost of the property of the public utility is the proper figure to be used, and the other that the present value of the property is the one to be used. Mr. Dow's statement of the problem is not sufficient. A better statement is as follows: Wanted an appraisal for the purpose of determining rates, or to determine the amount of securities to be issued. Shall actual cost to date be used, or shall present value be used? The final answer has not yet been given. Also, the problem is not so simple as it appears from Mr. Dow's point of view. If it was, it would be unnecessary to discuss the matter before this Society, as he states when he says "A valuation is an engineer's work, and the manner of making it is properly a subject of discussion by this Society." What, under his premises, is there to discuss?

There is no confusion whatever in the writer's mind between "cost" and "value." This is all so clearly indicated in the paper, partic-

Mr. Gandolfo. ularly on page 2471*, under Heading 6, as to need no other comment. Mr. Dow must have overlooked this part of the paper. Cost is an actual stable figure, known and positive. The principal thing about value is that it is unstable, and no one knows precisely what it is, as has already been shown.

Mr. Dow thinks that an appraisal or valuation based on the actual cost to date could be prepared by a competent accountant. The writer has already shown the fallacy of such beliefs.

On page 3187† Mr. Dow quotes a sentence from the paper, and comments on it. This sentence is found in the early part of the paper, and unless the greater part of the paragraph in which it occurs is quoted, it has no particular force or meaning. It is not an argument, is not intended as one, and is simply an opening sentence leading up to a discussion.

On page 3188† Mr. Dow says:

"Are we not setting up a false standard permitting that an engineer's opinion of value shall be different when his client proposes to buy, from what it is when he proposes to sell? Is it not the duty of an engineer called on to make a valuation to say that he finds certain property the present value of which is thus and so?"

Then, in the very next sentence, Mr. Dow contradicts himself, and admits that the "engineer's opinion" of value may be different under different conditions, when he says:

"If the engineer's opinion is asked as to what would be a wise price to be accepted or offered for that property; or if he is appointed arbitrator to fix a price; or if he is asked to express an opinion as to a fair rate of return to be allowed on the property in a rate-making case—under any of these conditions, an opinion as to price or as to rate of return is proper; * * *."

All of this also directly contradicts Mr. Dow's own statement of the problem, as given on page 3187.†

Referring to the last two paragraphs of Mr. Dow's discussion—there is no false standard whatever set up for engineers by the actual-cost-to-date method of appraisal for rate-making and security issues; nor is there any necessity for any confusion in engineers' minds in regard to the question. The entire matter is one of logical reasoning and deduction. The question of appraisal is not one in which the engineer is to "inject his opinion," no matter what system is used, but is one in which he is to determine facts as he finds them. The only system that gives facts is the actual-cost-to-date system. The gist of the matter is that Mr. Dow has ignored the fact entirely that the public utility corporation is a creature of the State, and to a

* *Proceedings*, Am. Soc. C. E., for October, 1914.

† *Proceedings*, Am. Soc. C. E., for December, 1914.

greater or less degree is the agent of the State. It must also be admitted that public utility property is held under a very different tenure from that of other property. Otherwise, the very fact of State regulation of rates, of security issues, and of the very details of the business, would be impossible. Mr. Gandolfo.

Mr. Lavis, in opening his discussion, refers to the recent paper* by Mr. Alvord. This is not a discussion of Mr. Alvord's paper, and the writer regrets very much to have to refer to it, but, as it has been brought into this discussion, it is only necessary to say that the writer has read this paper, and finds that the same ideas seem to obtain therein as were set forth in a former paper by the same author, and were severely criticized by the writer.† The writer regrets that he has not had time to prepare a discussion of Mr. Alvord's second paper, setting forth in detail some criticisms of the statements embodied therein.

On page 3189‡ Mr. Lavis says:

"It would appear at once that a valuation based on the actual cost to date method, * * * would by no means necessarily give a true estimate of the 'present-day value.'"

On page 3190‡ he practically repeats this statement. The writer does not know of any one who ever claimed that it necessarily did. It is strange how the advocates of the present-value method of appraisal persist in accusing the advocates of the actual-cost-to-date method of confusing the two results. The two results may be and probably are entirely different. No one makes any claim that they may be or ought to be the same. This is all clearly set forth in the paper. Perhaps Mr. Lavis overlooked it.

On page 3189‡ Mr. Lavis says that it is often difficult to see why depreciation should be deducted from the reproduction cost of a machine that is giving 100% efficiency. If Mr. Lavis will refer to pages 2466, 2467, and 2468,§ he will find cogent arguments there set forth as to why this question of depreciation should be given very careful consideration, and how it should be treated.

Furthermore, it is perfectly possible for a "machine" (using this term in its general sense, as well as specifically) to continue to give 100% efficiency, or perhaps very nearly 100% efficiency, up to nearly the end of its period of usefulness. The question of efficiency may have nothing whatever to do with depreciation, which is constantly going on, and much less with obsolescence and inadequacy. Of course, in a large utility, such as a railroad, different parts are constantly

* "Fundamental Principles of Public Utility Valuation", *Transactions*, Am. Soc. C. E., Vol. LXXIX, p. 117.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 863.

‡ *Proceedings*, Am. Soc. C. E., for December, 1914.

§ *Proceedings*, Am. Soc. C. E., for October, 1914.

Mr. Gandolfo. being replaced. At the same time, there is an example in the New York, New Haven and Hartford Railroad, showing how far depreciation and obsolescence (which is only another form of depreciation) can be allowed to go, and still permit the utility to give 100% efficiency up to the time that the inevitable collapse occurred, which was bound to come, under the conditions of deferred maintenance which existed on that road.

The rolling stock of the trolley company previously referred to is another example as to why depreciation must be considered in any appraisal. As far as transporting passengers is concerned, the old ramshackle cars are giving 100% efficiency, just as new cars would do, but sooner or later (and probably sooner) these cars must be replaced. They are not worth 100%, or anywhere near it, and it would be misrepresentation so to consider them, either in an actual-cost-to-date or present-value method of appraisal. The question of "efficiency" has no place in a purely physical valuation. When intangible elements, such as "going concern" are considered, then it is time to take up "efficiency", but in a different sense from that in which Mr. Lavis uses the term.

On page 3190,* Mr. Lavis says:

"* * * some general method must be adopted which will be equally applicable and fair to all the railways of the country, to determine with a reasonable degree of accuracy and fairness their present-day value, * * *."

This is a bare statement of Mr. Lavis' opinion, without any supporting arguments or facts, and he is simply "begging the question" when he uses the term "their present-day value". This is the very thing that is under discussion, and Mr. Lavis has advanced no arguments to show why "actual-cost-to-date" is not just as fair, and should not be used in place of his words, as just quoted. Nor has he refuted any of the arguments of the writer showing why this method should be used.

Mr. Harte, in his opening paragraph, attempts to deny that there are very material differences, in some respects, between the public utility corporation and the private business. This is not substantiated by the facts. These differences are all clearly set forth in the paper on pages 2449, 2450, 2451, and 2475,† and at a glance it can be seen so clearly, from the very fact of detailed control of the public utility, its property, its acts, and the conduct of its business by the public utility commission of the State, or other controlling body, and it is also so well testified to by the various Court decisions in every line relating to public utilities, as to seem to need no further comment.

* *Proceedings*, Am. Soc. C. E., for December, 1914.

† *Proceedings*, Am. Soc. C. E., for October, 1914.

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Referring to the second paragraph of Mr. Harte's discussion, if he will refer to page 2445* he will find the following sentence: "Any business is susceptible of endless modifications and extensions, * * *." This he evidently ignores. "Wildcatting" is just as possible in a private business, and is just as reprehensible, as in any other, but there is this great exception, compared with the corporation: Such business methods cannot be hidden behind securities, as they can be in the stock company, and innocent people brought into the matter without their knowledge and consent.

Mr. Harte is absolutely mistaken when he says, on page 153,† "Proportionately, the private business deals as extensively with that portion of the public in its field as either of the other groups; * * *." Why the private business does not, and cannot, is so well set forth in the paper as not to require repetition or further explanation; also, his statement that the "liability is proportional to the holding", is not borne out by the facts. Everything the owner of a private business has is behind that business. If he has invested $100 000 in a private business, and his personal assets are $1 000 000, all of this can be levied on in case of failure; but if he held $100 000 of securities in a company, this would be all that he would be liable for, no matter what his personal fortune was. Mr. Harte admits this fact on page 154† when he says:

"* * * modern business demands such large capital that few care to put so large a part of their money in one investment, but would rather divide it among many projects, in order to reduce the extent of the loss in case of failure of one or more."

All of this was fully set forth in detail by the writer on page 2446.*

Mr. Harte's third paragraph is not very clear, but the writer supposes he meant to say that if an old concern wished to expand and build additions, and estimated the cost of such additions as the same as the original cost of the old plant, instead of basing it on the present value of labor and materials, that they would very likely fail. Of course, they probably would. If any one did anything as foolish as that, he would deserve to fail; but what this has to do with the subject, or what point in physical valuation it elucidates, is not easy to see.

On page 154† Mr. Harte refers to the use of the roads by teams. Mr. Harte might as well say that if a man calls a policeman to arrest a thief, he is asking a favor of the State, or of the municipality. He is not; and neither is the teamster or firm that uses the road. In cases of this kind, the citizen is exercising his right, for which he pays in taxes, and is asking no special favor of the State. Marquee and

* *Proceedings*, Am. Soc. C. E., for October, 1914.

† *Proceedings*, Am. Soc. C. E., for January, 1915.

Mr. Gandolfo. area-way permits, etc., also mentioned in this paragraph, are not special privileges, but are such as any citizen, in like circumstances, may obtain. Applications for these by individuals are not in any way connected with the special exclusive favors from the State under which a public utility corporation does business.

The writer wishes to assure Mr. Harte that he is throughly familiar with the law of eminent domain, and knows exactly why it was necessary to enact such a measure. The writer did not say it was a special privilege to further the affairs of a corporation. What he did say on page 2445,* in referring to the private business, was, "Nor can it invoke the right of the law of eminent domain to further its own affairs".

On page 154† Mr. Harte says that a partnership is nothing more than a corporation of special form. This is not true in the accepted meaning of the words "partnership" and "corporation", both generally and legally. The only sense in which it can be said to be true is in an economic one, and this is not the way in which Mr. Harte used these terms.

On page 154† Mr. Harte states that the majority of the security owners of a utility control it. This is only so theoretically, and it is by no means so practically. When the securities of a public utility are widely scattered among perhaps thousands of stockholders, it is a physical impossibility for many of them to attend meetings, even if inclined to do so. Therefore it is absolutely impossible to secure the attendance of anything like a majority, or even a very large percentage of stockholders at any such meeting. Proxies are obtained by the board of direction or other managers of the utility, and thus the entire control of the business is often kept entirely in their hands for years. Does Mr. Harte think that a majority of the stockholders of the New York, New Haven and Hartford Railroad agreed to the business methods that led to its undoing? Or that a majority of the stockholders agreed and gave their sanction to the methods of conducting business as followed by the 'Frisco Lines, or the Rock Island Lines?

When Mr. Harte says, on page 154,† that Principles 2, 3, and 4, as given on page 2449,* are as characteristic of private business as of corporations, he fails to consider the difference between the methods of borrowing money, generally on notes, followed by private business concerns, and the method pursued in lending money on securities as collateral, which latter may not mean, and generally does not, that the concern whose securities are borrowed on is borrowing the money at all, but some outsider is raising the money for something entirely different. This matter of raising money on securities could be gone into at great length, but a glance at the entire matter is sufficient to

* *Proceedings*, Am. Soc. C. E., for October, 1914.

† *Proceedings*, Am. Soc. C. E., for January, 1915.

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show the radical difference between this and the borrowing by a private concern.

The writer fails to see where the Sherman Act and the cases decided under it, mentioned on page 155,* have anything to do with the question under discusion.

In the literature on the subject of physical valuation, Reasons 1 and 2, on page 2453,† are two of the three most often advanced by present-value advocates of appraisal. They were not gotten up by the writer purely for this paper, as the wording of Mr. Harte's remarks would seem to indicate. If they are not correct reasons, it is the fault of the present-value advocates and the judiciary.

In regard to Court decisions, the writer does not ignore or deny them, as is set forth on pages 2451 and 2453†; but Court decisions do not prevent engineers from striving for the right, and doing all in their power to bring about just and equable solutions of questions that are of vital interest to the State. If Court decisions are the only ground on which present-value advocates can base their case, as Mr. Harte hints in discussing Reason 3, then indeed is their case weak. For, if a premise is ultimately found to be unjust, even though supported by Court rulings, it must ultimately be abandoned, and just and stable ones substituted.

As to the "general rule", the writer is familiar with this expression, but sees no reason to change any of the wording on page 2455† in regard to it.

Another reason advanced by Mr. Harte for the present-value method of appraisal is that it warrants the professional existence of engineers. If this is one of his arguments, then indeed is his case still more weak. For it is one of the fundamental ethics of the Profession, if the engineer finds he is not needed, to say so, and not try to "make a case", as is so often done by a certain class in the legal profession. Be this as it may, it has already been shown that engineers are just as necessary for the actual-cost-to-date method of valuation, as for any other.

The actual-cost-to-date method does not deprive enterprise of its reward, nor does it accept the "unwarranted expenditures of the dreamer", as is all set forth in full detail on pages 2452, 2453, 2459, 2469, and 2470.† Mr. Harte's remarks in the last paragraph on page 155* do not appear to the writer to be consistent with the facts.

On page 156* Mr. Harte quotes from the Minnesota Rate Cases.‡ Referring to this case, the Court refused to allow the reproduction cost of real estate, as is particularly set forth by Justice Hughes on pages 44, 45, and 46.‡ Furthermore, on page 45‡ Justice Hughes says:

* *Proceedings*, Am. Soc. C. E., for January, 1915.

† *Proceedings*, Am. Soc. C. E., for October, 1914.

‡ Senate Doc. No. 54, 63d Cong., 1st Sess.

Mr. Gandolfo. "The cost of reproduction method is of service in ascertaining the the present value of the plant, when it is reasonably applied and when the cost of reproducing the property may be ascertained with a proper degree of certainty. But it does not justify the acceptance of results which depend upon mere conjecture."

Now, if it is wrong to appraise or value the land of a public utility at its reproduction cost (as Justice Peckham also hinted in Willcox *v.* Consolidated Gas Co.), is it not wrong to value any of the property at reproduction cost? As far as this is concerned, is not this decision contradictory? How can one rule apply to one kind of property, and another rule to another kind? It seems as if the Supreme Court began to realize the injustice of the reproduction-cost method of appraisal, but on account of the "general rule" was loath to say so.

Of course, the writer, in an actual appraisal, does not advocate basing any figure on "an old print". This, of course, would be absurd. The "old print" was simply used as an illustration in an argument; but, very often such things are of great value in an investigation.

In regard to trestling, the writer uses the term "temporary trestlework" and to this extent Mr. Harte has misunderstood him. Of course if a permanent trestle has been replaced at a higher cost, this is the figure that should be used. This is covered by Item 7 on page 2452,* which Mr. Harte evidently overlooked. Therefore his statement that, according to the writer's argument, such new material would be charged to operation, is not correct.

In regard to Mr. Harte's arguments about purchase and sale, these in no way change the fact that a public utility should be valued at actual cost to date for rates and security issues. To begin with, a public utility cannot sell or transfer its property, without the express permission of the State. "* * * the property so used is charged with a public trust and is devoted to a public purpose. Such property is dedicated irrevocably to the performance of this trust due the public and for its benefit and that of the inhabitants of the municipality." (Pond.) A study of Court decisions shows this. Now suppose that the actual cost of a utility has been $1 000 000, including all items, and that a rate of return of 8% has been allowed, giving a net yearly profit of $80 000. Now suppose that the present value of the property is $1 500 000, or that it has increased 50% in value. This in no way affects the original cost. If a purchaser now comes along, and after permission to sell is obtained, he buys this plant at this figure, the income would still be 8% on $1 000 000, or 5⅓% on $1 500 000. The new owner has bought with the express knowledge of what he is going to get and why he is going to get it. The actual cost to date is not changed, the rate of return is not changed; and, being a public utility,

* *Proceedings*, Am. Soc. C. E., for October, 1914.

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and agents simply having been changed in a practical sense, there is no reason why the rate should change. Thus no injustice is done to any one interested in the transaction.

In the last paragraph of his discussion Mr. Harte speaks of a "free sale" of a public utility. There is no free sale whatever for public utility property and never can be. The fact of there being no free sale, either from a market standpoint, or from a legal standpoint, for such property, is one of the facts that many present-value advocates seem to wish to ignore.

Mr. Green says that the principles of valuation must satisfy the requirements of law, of economics, and of engineering. This is true; but it must be remembered that two of these sciences are not positive, and the human element enters very strongly into them. The only one of the three that is positive is engineering, and this one only when all conditions relating to the particular problem under advisement are absolutely known. As far as appraisal and valuation are concerned, this branch of engineering is less positive than any other, and partakes more of the nature of speculation and surmise, unless the actual-cost-to-date method is used. Then all unknown elements are eliminated to a greater degree than by any other method.

As for principles of law, these are mostly man-made (as Mr. Green evidently refers to common law and statute law, and not to Nature's laws), and are constantly changing. Law is supposed to embody principles of right and wrong, but even these are purely relative terms. The code of ethics of the head hunter of the Philippines is entirely different from that of the people of the United States, and his customs and laws are therefore entirely different. Many ancient and medieval laws are now looked upon by us as having been harsh and cruel, and yet the people of those days thought nothing of them. According to his code of ethics, the engineer must keep constantly in mind what is right and wrong, and strive in every way to advance the right.

The writer wishes to assure Mr. Green, as well as he has Mr. Harte, that he thoroughly understands the law of eminent domain, and just why it was necessary to evolve it. The fact remains that the private individual cannot resort to this law, to further his own affairs. If resort is had to this law, a public necessity must be shown. The power of the law is not delegated to any one. Only the State can exercise it. If the law is to be involved, recourse must be had to the Courts. The public utility or "carrier" cannot go to any one who stands in its way, and say we want your land, or your buildings, or your property, and simply take them. This is what the remarks on this subject, as Mr. Green puts them, would seem to indicate could be done. Condemnation proceedings can only be carried on through the Courts.

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The writer fails to see what connection there is between one road going to the Courts to condemn a right of way across another road and the method to be used in a valuation.

Referring to Mr. Green's remarks on watered stock, this may be "faith", but is more apt to be "fraud". As for the farmer who asks \$20 000 for his land, he does not and cannot ask the public to pay an income on his "watered value"; but, when watered stock is issued, it is expected that dividends will be paid thereon, and it has already been shown how such stock becomes a burden on the public.

Mr. Green does not realize the function of government when he speaks of fools and their folly on page 159.* The fact is that a large part of any government is organized for and occupied with the express work "to protect fools from their folly", or, to put it in different words, to protect honest people from the pitfalls and machinations of the dishonest elements of the population. It is perfectly true that all governments, in undertaking this work, "have a bigger job than the Panama Canal."

Bread, clothing, and recreation are certainly vital to a community. But a bakery, a haberdashery, or a picture show is not. If any one of these is not satisfactory, or if satisfactory service is not given, there are hundreds of others where the consumer can go and get satisfactory service. If one is destroyed, or fails, or goes out of business, the community at large is not affected in the least. The patrons of this one distribute themselves among others of like kind.

With a public utility, however, this is impossible. As it is a monopoly, the public must patronize the utility or go without. Therefore Mr. Green's criticism of the writer's definition is apparently not well founded.

It is not a question of one law for one enterprise and a different law for another kind of enterprise. It is a question of having a just law, impartially enforced, applying to all enterprises that exist under certain conditions and circumstances.

The writer has carefully examined pages 2451 and 2452,† and fails to find any contradictory statements whatever thereon. One sentence does not make a book, or an argument.

To show why a valuation cannot and should not necessarily be the same for all purposes, no better example can be given than that of the recent appraisal, by the Comptroller of New York State, of the Hales Bar development on the Tennessee River. Here was a development which had cost more than \$10 000 000; which should not, if reproduced, cost more than \$5 000 000; and which was in no sense a going concern, and with no immediate prospect of being one. To appraise this develop-

* *Proceedings*, Am. Soc. C. E., for January, 1915.

† *Proceedings*, Am. Soc. C. E., for October, 1914.

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ment at actual cost, for the purpose of the inheritance tax, would certainly not be fair to the owners. To appraise it at present value would be meaningless, because, strictly speaking, as the plant stands, it has little more than scrap value; but, as the heirs signified their intention of carrying on the development, this plant certainly had some value for purposes of taxation, as was finally decided when its inheritance tax value was fixed at $3 200 000.

As to a valuation for taxation, it has already been shown on page 2473* that it makes no difference to the utility or the rate-paying public whether such a valuation is high or low. The entire matter is not a question of low or high valuation, as Mr. Green puts it, but is one of a just and equitable appraisal and valuation to all concerned.

As far as the acquirement of land goes, it is an impossibility for any public utility to look far enough into the future to meet all its requirements. Furthermore, granting that it could, what is the limiting period in the future for which present generations may be taxed in order to provide for future generations' needs? How much unproductive investment is a public utility justified in making, for future needs and developments, and how much is it justified in asking present generations to contribute toward such investment? Very little, if Court rulings are an indication of anything in this line.

The writer does not criticize roads which are now improving their lines and grades or making other betterments, as Mr. Green says he does on page 2462.* Referring to this page, it will be found that the writer says such changes must be paid for out of profits, if they take the place of old work. This same matter of additions and improvements is also covered by Items 7 and 14, on pages 2452 and 2453.* Mr. Green, in the same paragraph, links the question of excess size of plant with this subject. What possible connection there is between the two, the writer is unable to see. Betterments and improvements certainly have no connection with excess size of plant. This question of excess size of plant has already been covered in discussing the future requirements as to land, and the same arguments apply to the entire plant. Therefore there is no meaning in the last two sentences of Mr. Green's discussion, as applying to the principles of appraisal and valuation as advanced by the writer.

At the very beginning of, and all through, his discussion, Mr. Mayer persistently says the writer's ideas are socialistic. The writer has already referred to this matter, has disclaimed any leaning toward socialistic doctrines, and wishes again to emphasize this fact.

In his first paragraph, Mr. Mayer says, "The idea that compensation should be in proportion to the effort made to secure it, is the leading motive of his paper, * * *." However, there is nothing in the paper

* *Proceedings*, Am. Soc. C. E., for October, 1914.

Mr. Gandolfo. to indicate this. If Mr. Mayer will refer to pages 2452 and 2453* he will there find fourteen items which cover all things to be taken care of in a physical valuation. There is nothing socialistic here. On pages 2457 and 2458,* in discussing an enterprise that has failed, there is absolutely nothing socialistic in the treatment of this subject. Again, on page 2459,* in discussing promoters' fees, there is nothing of a socialistic nature, and on page 2470,* where the writer says a return should be allowed over and above the ordinary rate of interest return, there is certainly nothing partaking of socialism.

A public utility is not merely the tool of a private party for obtaining a revenue. It is primarily a tool to serve the public, and must be run and managed with this end in view. It thus differs widely in this respect from the private business.

On page 426† Mr. Mayer says: "Therefore, those who attempt valuations of complicated enterprises by ascertaining their cost of production meet with various unsolvable problems, * * *"; but he neglects to give any list of such unsolvable problems.

On pages 426 and 427† Mr. Mayer goes into a lengthy argument on the subject of competitive enterprises, and attempts to explain at length some matters connected with such enterprises. Mr. Mayer does not appear to realize, that with the vast majority of such enterprises there is no competition whatever; all arguments and reasoning in relation to them based on any such competitive theories are founded on false premises, and are therefore fallacious. This is the principal point in the entire matter, as stated by the writer at the top of page 2451*; and any one who does not understand and admit it, is simply groping in the dark.

In the last paragraph on page 426† Mr. Mayer says a mere percentage on the cost of an enterprise cannot be said to be any criterion as to the compensation for promoters and bankers. This is exactly what the writer says in the last paragraph on page 2459,* and shows on this account why such items should be put in at "cost", not at a "fair value".

On page 427† Mr. Mayer says: "Why not guess the total value without going to the trouble of ascertaining first the physical value?" This remark is so ridiculous, on the face of it, as to need no further comment.

On pages 427, 428, 429, and 431,† Mr. Mayer refers to what he calls a "guessed at" going value, and says the writer advocates this. On page 427† he is talking about competitive enterprises, and says: "The author thinks that an arbitrarily chosen or guessed at going value should be added to the physical value in order to obtain the total value of an enterprise." The writer did not say anything of the kind in rela-

* *Proceedings*, Am. Soc. C. E., for October, 1914.

† *Proceedings*, Am. Soc. C. E., for February, 1915.

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tion to a competitive enterprise, as a reference to pages 2443, 2444, and 2469* will show. The word "guess" in this connection has been created by Mr. Mayer. In the case of a public utility, where the question of rates is to be determined, will Mr. Mayer please inform the writer, and the Engineering Profession at large, how else the going value can be determined except by a more or less arbitrarily chosen figure, founded on a careful study and investigation of the particular problem under consideration? This, the only method there is, is in no sense a "guess", in the meaning of this term as used by Mr. Mayer.

In discussing this question on pages 428 and 429,† Mr. Mayer uses such terms as "* * * same profits as are obtained in competitive enterprises * * *", "fair market value", and "competitive profits". As has already been pointed out, none of these things exists in a public utility which is controlled by the State.

Now assume, as Mr. Mayer does, that a public utility has its rate fixed. This gives the future net earnings with a reasonable degree of accuracy. Then suppose these net earnings are taken as the going value (not being any competition, there can be no competitive earnings) and are added to the physical value to obtain the present or fair value. Then rates must be allowed on this new "fair value", and so on, *ad infinitum*. The fact is, that every one, the Courts included, after a careful study and investigation of each case, has been compelled to arrive at going value for a public utility by what Mr. Mayer has been pleased to call a "guess", when the question of rates has been under advisement. A study of legal decisions shows this. (Knoxville *v.* Water Co., 212 U. S., 1; Cedar Rapids Water Co. *v.* City of Cedar Rapids, 118 Iowa, 234, 91 N. W., 1081; Cedar Rapids Gaslight Co. *v.* City of Cedar Rapids, 223 U. S., 655; Cumberland Tel. and Tel. Co. *v.* City of Louisville, 187 Fed., 637; and many others.) To say that the revenue is to be capitalized, when the revenue is the very thing to be determined, is "begging the question", and is a farce on the face of it. Mr. Mayer's reasoning along these lines is fallacious.

On page 428† Mr. Mayer says: "He also claims to show that the same principles should be applied to the valuation of the property of a private individual," and then mentions a buyer and seller. Just what "principles" he refers to here is not clear. Furthermore, the writer was not applying any course of deductions to a private business, but was showing the development of the public utility corporation from the private business. As for the "sane" buyer and seller, all this is set forth on pages 2443 and 2444* exactly as Mr. Mayer puts the theory, only very much more in detail. The writer feels sure that Mr. Mayer must have entirely overlooked these two pages.

* *Proceedings*, Am. Soc. C. E., for October, 1914.

† *Proceedings*, Am. Soc. C. E., for February, 1915.

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The writer begs to tell Mr. Mayer that he is not "obsessed" by the "theory" that intelligence deserves no consideration in a valuation, as a glance at pages 2452, 2459, 2469, and 2470* will show. The writer supposes Mr. Mayer overlooked these pages.

The writer did not go into any extended detail as to how the actual cost to date of a public utility is to be obtained, for the very simple, reason that his paper is an argument as to why actual cost to date is to be used, not how it is to be obtained.

In the last paragraph on page 429,† Mr. Mayer says the writer's notion as to how profits are obtained in a competitive business appears "preposterous." Then he goes on to show how such profits are secured, all of which is exactly as the writer puts the matter on pages 2443 and 2444,* particularly the first paragraph on page 2443,* only much more in detail. In other words, Mr. Mayer says the writer's "notion" in regard to this is "preposterous", and then goes on to give, as his own, ideas exactly what the writer has already said. This is somewhat peculiar. On page 2469* the writer tells in a general way how a profit is figured in a competitive business. Mr. Mayer must know that in nearly every business, especially a manufacturing business, which corresponds more nearly to that of a public utility business, careful detailed costs are kept of every item entering into the finished product (both material and labor items), so that the actual cost of the product may be known, a profit made by adding a percentage to this cost, and the said cost of production watched and reduced wherever possible, for purposes of competition.

Another thing Mr. Mayer has entirely overlooked is the fact that on page 2445* it is stated: "Any business is susceptible of endless modifications and extensions * * *", and again on page 2474,* "* * * so also every public utility is capable of endless modifications and variation * * *."

On page 430† Mr. Mayer says a State that engages in business is interested in the profits or losses resulting therefrom. A State is interested in the losses, but is not interested in the profits. A State goes into business to supply some particular thing to its citizens at large, not to make a profit. Further, he says here the cost to date must be ascertained. As the owners of a public utility are practically only the agents of the State, then the cost to date of the enterprise is the figure to be ascertained, on which to calculate the profits, the profit in such a case being the compensation of the agent.

On page 430† Mr. Mayer says the writer's reasonings are quite indefinite, on account of lack of definitions for the terms used. The writer did not intend to use terms or words in any sense

* *Proceedings*, Am. Soc. C. E., for October, 1914.

† *Proceedings*, Am. Soc. C. E., for February, 1915.

other than that of their generally accepted meaning, and believes they all are perfectly clear. Mr. Gandolfo.

On page 431* Mr. Mayer talks of "competitive profits", in relation to a franchise of a public utility. There is no such thing as competition in any form in connection with a public utility (except in a very few rare and exceptional cases that can be ignored), as has been shown several times. Mr. Mayer seems to be "obsessed" by this idea of competition in connection with public utilities. His remarks based on this are meaningless.

His elaborate formulas on page 431* have also been shown to be meaningless, so far as they possess any value for determining the value of a public utility when rates are to be determined. To capitalize the revenue, when the revenue is to be determined, has already been shown to be an impossibility and a farce.

Mr. Mayer says that he believes the paper, as a logical product, is without value (page 428*); that its definitions are not clear, and that it does not give an accurate description of any method of valuation (page 432*), and also states that criticism should be without reserve, when one succeeds in getting a hearing before a scientific society (page 432*). The writer is willing to let his paper itself answer such criticisms.

The fact of the matter is, Mr. Mayer shows very conclusively that he has not read the paper carefully, even for the purpose of his own discussion, much less to have studied the lines of reasoning as therein followed to their logical conclusions.

Conclusions.—Before concluding this argument, the writer wishes to call attention to the fact that in two discussions he has been misquoted. The writer believes that mis-quotations are inexcusable, particularly in a scientific discussion.

In attempting to refute any line of argument, it is necessary to follow it step by step, showing if possible at every point wherein it is fallacious and then setting forth at the end wherein the conclusions are erroneous. It is then necessary for the discourser to give arguments in his turn showing why his premises are the correct ones, following this with his conclusions, founded on his premises. This none of those who have discussed this paper has done, or attempted to do. Each one has picked out an item here and there, and has attempted to show that it is not founded on sound principles of reason and justice, without giving any heed whatever to the main subject matter of the paper, and what preceded and what followed.

The writer has taken up these arguments in a general way, and then has taken up each discussion in detail, and has shown wherein they in turn are not founded on sound principles of reasoning and justice.

* *Proceedings*, Am. Soc. C. E., for February, 1915.

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Such expressions as "* * * it seems only necessary to refer briefly to the matter in this discussion in order that Mr. Gandolfo's statements may not become part of its official records without some protest", quoted from the discussion by Mr. Lavis; "* * * one of the few rays of bright light in the fog in which so many writers have concealed the facts", from that of Mr. Harte; "Although the writer believes that the paper, as a logical product, is without value"; and "The author's notion as to how profits and values are ascertained * * * appears preposterous * * *," both taken from that by Mr. Mayer, are not arguments (and such remarks never can be), and do not in any way refute the statements and conclusions of the writer.

Therefore, in the absence of any logical arguments or reasons advanced in refutation of those in his paper, the writer must conclude that his premises are correct, his reasoning logical, and his conclusions just and true.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

WATER SUPPLY OF THE SAN FRANCISCO-OAKLAND METROPOLITAN DISTRICT

Discussion.*

By Messrs. Rudolph W. Van Norden and Burton Smith.

Rudolph W. Van Norden,† M. Am. Soc. C. E. (by letter).—This paper has been commented on, and criticized by, engineers, either with extreme favor, or with disfavor amounting to condemnation. Favorable criticism has emanated from those engineers who are familiar with the San Francisco water-supply problem, but who have not contributed to its solution; also from those who, at some time, have contributed to the study of the problem, but are not now engaged in the development of the Hetch Hetchy scheme to the exclusion of all others. The latter group naturally covers the majority of engineers more or less familiar with this subject. The adverse criticizers comprise only those engineers who are engaged on the development of the Hetch Hetchy plan, and are thus not interested in any consideration or argument which might jeopardize the fulfilment of that project. Mr. Van Norden.

Mr. Cory's paper cannot be considered as a cut-and-dried academic technical article of the orthodox variety, which engineers have been taught to expect in the publication of technical societies. From this view-point there is no doubt that the adverse criticism by Mr. O'Shaughnessy is in order; but the time has long since past when papers and their discussions, having as a purpose the accomplishment through engineering science of some great economic improvement, can be limited to mathematical formulas, available data, and deductions from reasonings within narrow limits, where such reason-

* Discussion of the paper by H. T. Cory, M. Am. Soc. C. E., continued from August, 1915, *Proceedings*.

† San Francisco, Cal.

Mr. Van Norden. ings are based on premises which are but sage theories, in reality little better than guesses.

Engineering science is becoming recognized more and more as of infinite breadth, rather than infinitely circumscribed by rules and theories. Economic questions, in themselves apparently outside of the radius of action of the civil engineer, questions of law and personal rights, equally foreign to the old-time ideas of the duties of the civil engineer, all enter into the solution of a problem so complicated and so vital to a commonwealth, as is the water supply in question—not as foreign intrusions, but as essential parts of an engineering problem. Civil engineering, in its highest sense, does not, alone, mean the mere design and construction of dams, tunnels, bridges, or power-plants—given unlimited capital and unlimited time, almost anybody can accomplish such things—but it does mean the selection of ways and means of doing the greatest good for the advancement of mankind with the greatest lasting economies.

In the San Francisco water-supply work, much deep and thoughtful research has been made. It is not true, as has been often stated, that the people's money spent in this work has been wasted. There has undoubtedly been much time and some money spent in "offside play", and irrelevant actions with political savor, but this has had its value in awakening public interest to action.

The problem, as attacked by Mr. Grunsky, the City Engineer in 1902, was handled, within the limits of cost and time available, in a masterly manner. For the object in view, as it was then understood, Mr. Grunsky's report was a sound beginning; but time has proven that there were lacking, and unknown at that period, the very elements which are shown to exist by the study in Mr. Cory's paper. As economic conditions have become better understood and realized, as the community has developed, and as settlement in the State has increased, much work toward arriving at a basis for obtaining proper results has been done. This has all been creditable, but the problem is a far bigger one than was at first realized, and all this preliminary work has been necessary to obtain insight into the matter.

Finally, the plans which have now been advanced for a proper development of a water-supply system were worked out with the object of coping with the problem in its broadest form. Theoretically, this has been a step in the right direction, but the criticism which has been freely offered is that the gauge adopted has not been broad enough. In order to make the suit fit, the cloth should be selected of ample width and breadth. In this case, the attempt seems to be to use the cloth at hand and accept a suit which it will make, with the grave possibility that it will not fit, or will be too tight.

The day is past when one engineer can determine the solution of a broad engineering problem by stating that, in the wisdom of his

opinion, thus and so are truths, and expect the matter to be settled without further proof. The statement has been made in the criticism of Mr. Cory's paper that the plans for the development of a San Francisco water supply, engineering, economic, industrial, and financial, have been carefully made and settled, and, as an excuse for this statement, a further suggestion is made that this has been decreed by engineers of prominence. Mr. Van Norden.

There is ample evidence that a very considerable quantity of creditable and serious preliminary work, as a means toward an end, has been done, but Mr. Cory's paper goes to show that the pure and proper engineering work necessary for the solution of this problem is by no means completed. The paper illuminates the many pitfalls to be encountered, and points to the invisible angles from which views of the problem must be made, without the consideration and treatment of which all previous efforts toward its eventual consummation must be nullified. If this is true, and many engineers believe it to be so, then Mr. Cory's paper is a pure engineering document of the highest order.

Within the last three years, the present civic administration, through the city engineer and the city attorney, has promoted a number of public enlightenment campaigns, or, what might better be termed, a single campaign with intermittent periods of great activity. The noticeable feature of this campaign throughout has been, not to instruct the populace as to its efforts and accomplishments in the preparation for the development of the best plan for a water supply as a broad engineering and economic proposition, but, rather, as to its plans for a reasonable engineering development of the Hetch Hetchy project. The administration has felt itself justified in taking this attitude from the fact that some years ago the City of San Francisco purchased reservoir sites on Cherry Creek and at Lake Eleanor, spending in this purchase considerably more than \$1 000 000, and, in 1910 the people voted to bond the city to the amount of \$43 428 000, for the construction of the Hetch Hetchy project. Thus this project has been tenaciously upheld as the "best development possible", with little regard to the feasibility of other suggested plans, some one of which might fulfil the conditions to be met in a more satisfactory manner at a less cost to the people.

Much that has been said in this campaign of education has been sound, but there has also been much said with the intent of befogging the issue, and many statements have been made which could not possibly have any basis in fact. The right statements, and those which it would be difficult to substantiate, become apparent on examination to any engineer or investigator, but the many vital points which have been left out and might seriously cloud the appearance of superiority of the Hetch Hetchy source over some other have generally formed

Mr. Van Norden. grounds for contention, and have resulted in the deadlock that seems to exist at the present time.

What should have been discussions of interest and value have generally been vicious attacks of a personal nature from both sides. Nor in the discussion of this paper have personal aspersions been absent; and these are the things which are not only out of place in a scientific publication, but are inexcusable.

There have been a good many engineers who have become familiar with the situation in a general way, and who would gladly join in discussions with the object of arriving at an absolutely true and unbiased basis for the correct solution of the problem. Of these, some are entirely independent and free from any interests touching this water-supply situation, others are more or less interested through clients, and still others are more directly interested; but, unfortunately, all the engineers thus situated, who have not made some expression of their studies or opinions, are divided into two classes. In the first class is the man who does not wish to take issue with the administration, because he feels that he may jeopardize his chance of working into the development in a professional capacity. In the other class is the engineer who does not wish to be accused of being in the employ of some interest whose plans conflict with those as announced by the city engineer.

Throughout this campaign all those who have dared to offer any opinions contrary to those of the administration, or to disagree with, or to take any action which would block the plans of, the administration in its procedure with the Hetch Hetchy project, no matter who they are, or what their standing, have been flatly accused of being in the wrong from some ulterior motive. This accusation has generally taken the form of statements to the effect that such persons were working in the interests of companies, or individuals, with plans for water-supply developments inimical to that of the Hetch Hetchy, and that such persons were receiving pay for their contrary opinions. There was a long period when any engineer daring to express a contrary opinion was said to be in the employ of the Spring Valley Water Company.

Such campaigning has undoubtedly had an immediate desired effect, but, in the long run, it has caused many people to think, and in the end, the administration cannot but feel the ill effect of these short-sighted statements, where there has been so little foundation in fact.

In proceeding with the plans for the Hetch Hetchy project, the administration has taken its position—which it has apparently determined not to compromise—on the basis of two engineering reports. These are: the report of John R. Freeman, M. Am. Soc. C. E., which includes several reports on alternative projects by other engineers, and the report made on the order of the Secretary of the Interior by

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an Advisory Board of Army Engineers. Since the acceptance of these reports, the city engineer has done much work in amplifying and recasting the information embodied, with respect to the Hetch Hetchy project, until his part of the work is probably more valuable than the contents of these two reports combined.

In the earlier work done by the office of the city engineer, namely, during 1902 and 1908, a project was outlined for the immediate development of a supply of 60 000 000 gal. daily, with provision for doubling that quantity. On this basis a system was designed which would compare favorably in cost with that of all other projects, and held no very serious engineering difficulties.

These plans represented sound practice in engineering construction. Furthermore, the water-shed and storage system proposed offered an ample reserve which took cognizance of all complications, and would satisfy adverse claims to water.

The present administration realized the necessity for a water system capable of delivering 400 000 000 gal. per day. This requirement presents a very different engineering and economic problem, compared with which, the first plans are as mere child's play.

With the determination of the necessity of a greater quantity, there was immediately presented an array of possibilities for the development of some other water supply, which, under the smaller draft of the earlier plans would be uneconomical, and hence unfeasible, but, under the new plan of increased output, might prove to be much superior to the Hetch Hetchy project.

These possibilities were apparent to every engineer and investigator who knew anything about the water situation, but, as has been said before, the inflexible stand taken by the administration precluded discussion, or even opinion to any extent.

The Freeman report is, on its face, a justification of the use of the Hetch Hetchy Valley and the Tuolumne water-shed. It is the suit made to fit the cloth. As a mass of statistical information, it is a splendid compilation, and, considering the comparatively meager data on which it was built, is an apparently strong presentation of the subject. The writer has discussed with many other engineers, the general engineering plan adopted in the Freeman report, for the purpose of determining the soundness of the reasonings held therein and the plausibility of cost estimates, and to obtain, if possible, suggestions for improving the plan, or lowering the cost. Though there are diverging opinions on these points, the writer believes that the general plan for pressure tunnels, as outlined in the Freeman report, is visionary, and that the estimates of cost are entirely too low. In this opinion he has been upheld by other engineers. The writer does not take the stand that the general plan is untenable, as it is possible to give much latitude in the final designs and determina-

Mr. Van Norden. tions, but unless this is possible and unless the foregoing statements are disproved, the entire plan for the development of so great a transportation as 400 000 000 gal. of water per day becomes practically unfeasible in the face of possible supplies from other sources.

The writer is strongly opposed to the construction of unbalanced pressure tunnels. By this expression is meant, a tunnel operating under hydrostatic pressure where the mass of ground over and around the tunnel is depended on to resist the tendency toward bursting, and where there is no hydrostatic pressure from the outside which would tend to balance the pressure from the inside. A balanced and hence proper use of pressure tunnels is illustrated on the Catskill Aqueduct in the crossing under the Hudson River and the tunnel under the City of New York.

In the unbalanced pressure tunnel there is always a tendency toward leakage, depending on the homogeneity of the lining and surrounding structure. This leakage will work toward the surface, and the pressure follows in time. Theoretically, there will come a time when some layer of pressure close to the surface will overcome the resistance of the ground material, and a blow-out or slip will occur, only to be followed by another slip farther back. This action will continue toward the source of pressure until the entire structure of the ground fissures, or gives way bodily.

There have been, to the writer's knowledge, four pressure tunnels in California, all of which have failed in a greater or less degree, and in one manner or another. The two most notable and costly examples of such failures were the inclined unbalanced pressure tunnels in the Sand Canyon Siphon on the line of the Los Angeles Aqueduct. These tunnels were 9 ft. in diameter and were lined with plain concrete having an average thickness of 12 in. They were cut through granite. Both were inclined from the grade of the gravity conduit, these inclines being in the same direction as the respective slopes of the canyon sides, but with steeper inclinations. They were connected at their lower ends by a riveted steel pipe, 9 ft. in diameter, this pipe being run into and concreted in the horizontal portals of the pressure tunnels. The maximum thickness of ground, measured normal to the axis of the tunnels, was about 500 ft. in one case, and more than 100 ft. in the other. Both of these tunnels blew out shortly after they were filled with water. The symptoms were, first, a slight leakage, then a small movement, followed, in one case, by the tearing out of the whole mountain side. It was necessary to replace these tunnels by an all-steel siphon. Aside from the original cost of the work and the cost of rebuilding, the absolute stoppage of flow and use of the aqueduct, had it been in regular service at that time, would have been fully 2 months. The writer cannot conceive of any other result

which might have been expected from the use of such conduit construction. Mr. Van Norden.

In the plan for the Hetch Hetchy development, there are long stretches of pressure tunnel. In much of this distance, blow-outs or breaks would probably be impossible or unlikely, but there are undoubtedly many places where disasters of this sort might be looked for. One such break as was experienced at Sand Canyon would interrupt the flow of water for weeks, or even months.

Between the west side of the San Joaquin and the east rim of the Santa Clara Valleys, there are proposed, according to the Freeman plan, two pressure tunnels with an aggregate length of 30.7 miles. The first of these is to be 27.5 miles long. Geological studies of the country through which these tunnels are to pass show well-defined faults. A break or slip in the ground surrounding one of these pressure tunnels might be the cause of endless disaster to the system; and the emptying and refilling of the conduit, even for inspection or repair, might cause a most serious water famine in San Francisco by the interruption of the supply. Particularly would this be so if the present and proposed reservoirs of the Spring Valley Water Company, which are included as an integral part of the Freeman plan to act as stand-by storage, are not available for use in this connection; and, at the present writing, such an eventuality would seem to be probable, as the purchase of the Spring Valley system by the City of San Francisco has failed to carry at two elections.

The estimates of cost of the Freeman plan were based largely on unit cost data obtained from the work on the Los Angeles Aqueduct, but at a time prior to the actual completion of the work and the commencement of operation of that system. The writer believes that the use of these Los Angeles cost data in the Freeman report was premature and inapplicable to the Hetch Hetchy project; that in many points actual costs would be greater than those assumed; and that the total cost of the work, if it were possible to complete to successful operation in accordance with the Freeman plan, would sum up greatly in excess of the totals given in that report.

To sum up the preceding remarks, the Freeman report is by no means a complete solution of the problem to deliver water from the Tuolumne water-shed, either from an engineering or an economic standpoint, but it serves to disclose the magnitude of the problem of the development of the Hetch Hetchy project and to emphasize the complications unseen by the casual observer, and furthermore, to point out, not the ease with which the project may be constructed, or its cheapness, but rather, the tremendous difficulties of making a source —quite suitable for a small supply of 120 000 000 gal. daily—equal to the duty of the supply and transportation of 400 000 000 gal. daily, even for distant future consideration, which is a figure at the extreme

Mr. Van Norden. outside limit of the water-shed and storage possibilities, economic conditions not considered, and with little or no factors of safety.

The report of the Advisory Board of Army Engineers is an elaborate and very fair document, considering the variable sources of the data, many of which were of little value, and withal, comparatively meager, on which it was compiled. On a favorable interpretation of this report for the Hetch Hetchy project, the administration has placed much stress.

The report and subsequent extensions were used with good effect before Congress in assisting the passage of the Raker Bill. Few people have taken the trouble to read this exhaustive report through, much less to study it—an unfortunate condition, because it is broad in its conception, and shows much care and thought in detail.

The criticism of those who are familiar with it is that the administration, in using it, is careful to quote only those paragraphs, sentences, and even clauses, which will react favorably to the Hetch Hetchy plan. To study the report gives one the impression of a broader understanding of the subject by the Board of Army Engineers than the interpretation thereof which has been consistently maintained by the administration.

In placing those data before the Army Board on which to base a report, the city engineer was required to give, not only all data available on the Hetch Hetchy project, but also on the various other projects offered. This latter requirement was a difficult one, because the administration, pledged to the development of the Hetch Hetchy project, possessed only preliminary and incomplete reports already published on some of the alternative projects, and practically no data and little information on the remaining alternatives. It thus became necessary for the city engineer, within a very limited time, to examine these projects, collect data, and make reports of their possibilities, methods of construction, and costs. Attempts on the part of the city engineer were made to proceed with this Herculean task, which would require months where days were available. The results were naturally abortive and of little value, but served to accomplish in a measure the requirement of the administration, namely, to cause the Hetch Hetchy plan to stand in a superior light in comparison with all other suggested plans and projects.

As an example of the frailty of these reports made by the city engineer: two of the alternative projects suggested would be natural flow aqueduct systems, in many ways similar to the Los Angeles Aqueduct. One of these, known as the McCloud project, proposes to take a part of the natural flow from the McCloud River without the requirement of initial storage reservoirs as a primary requisite. The proposed line is to be a gravity-flow aqueduct, partly in canal and partly in tunnels, with the exception of the crossing under Carquinez

Strait, which would be a pressure tunnel similar to that of the Catskill Aqueduct under the Hudson River. Throughout its length of about 200 miles, this aqueduct would pass through a very rough and mountainous country—the eastern range of the Coast Range Mountains, bordering the west side of the Sacramento Valley. The greater part of this territory is uninhabited, and the aqueduct location, at the present time, is largely unapproachable by vehicles, because of the absence of roads. Mr. Van Norden.

In order to obtain data for the required report on this specific project, the writer was informed that the city engineer sent out a party which made a rapid topographical survey of the reservoir site at the proposed location of the diverting dam in the McCloud River. This required several days. No criticism could be offered for this part of the city engineer's work, except that the work and time spent on this part of the project were rather unnecessary. Another party, so the writer was informed, attempted to cover the 200-mile stretch of the proposed aqueduct line in an automobile, reaching its supposed position by means of an occasional intersecting road, and with the assistance of such portable instruments, including a camera, as would be used in rough reconnaisance work. Three days are said to have been spent in examining this stretch of country to obtain first-hand data for the compilation of the city engineer's report. The strenuosity of this work may be better realized when it is known that, for the greater part of the distance, there are no topographical maps, or accurate maps of any kind, in existence.

The writer is more or less familiar with the country through which this proposed aqueduct is to pass, and fails to conceive how any engineer, no matter how great his ability or experience, could properly cover this line and obtain data of any value whatsoever for a report in ten times the period taken by the party in question, even had he been over the country on some previous occasion. The proof of this contention is found in the reading of the report and the observation of photographs taken, purporting to be points on the aqueduct location, which are, in reality, several miles from its true position. As a further proof that a fair and unbiased presentation of this project was not made, the writer is not unconvinced that a better and cheaper means of bringing the McCloud water to the Bay Cities would be, not by a gravity aqueduct, but by pipe line through the open Sacramento Valley, operating under a static head only sufficient to overcome the friction head of the pipe. This feature was not treated seriously.

The writer is not in a position to state flatly that the McCloud project is superior in the broad engineering sense, that is, good engineering construction at lower comparative cost, better economic conditions, more equable personal rights, and the greatest good to the

Mr. Van Norden. greatest number, than is the Hetch Hetchy plan properly developed after exhaustive study; but it certainly represents many advantages over points now standing in the way of the Hetch Hetchy project consummation, and is, without doubt, worth very careful and unprejudiced engineering study.

The preceding statement is made absolutely without prejudice, as the writer has no interest for or against any plan for a water supply, outside of a desire to see the problem properly solved.

Those features which circumscribe the use of the Tuolumne River for municipal supply, and otherwise impose conditions which enter vitally into the solution of the problem as a whole, that is, the question of economics and personal rights, have been discussed in a limited way in connection with Mr. Cory's paper. The writer believes that there remain ample grounds for further discussion in a broader and fairer-minded manner on both sides, and these points must be all settled before plans for detailed construction can proceed intelligently.

Without going into any of these questions, or making the attempt to supplement Mr. Cory's researches along these lines, there is, however, one point which the writer desires to emphasize:

The City of San Francisco, through its representatives, made a hard fight before Congress for permission to construct reservoirs and conduits to carry water from the Tuolumne water-shed. This fight, which the people of San Francisco as well as the entire country were led to believe was a desperate contest between justice and right, on the part of the people, as against powerful and sinister interests whose only desire was to thwart the Commonwealth of San Francisco for exploitation and personal gain, resulted in the passage of the Raker Bill. This bill was, indeed, a hollow victory, for the adherence to its requirements loads upon the City added responsibilities of great magnitude. It also jeopardizes the water supply by limiting its output in years of low water yield far under the point of maximum use.

Coincident with the promotion of the passage of the Raker Bill, an independent report on the subject of the use of the Tuolumne water by San Francisco was asked for by the Secretary of the Interior, in response to Senate Resolution No. 191, from Dr. George Otis Smith, Chief of the United States Geological Survey. This report, in conjunction with a supplementary report by H. H. Wadsworth, M. Am. Soc. C. E., for the Advisory Board of Army Engineers, in response to the foregoing Senate resolution, is entitled, "Letter From The Secretary Of The Interior Transmitting, In Response To A Senate Resolution Of Oct. 31, 1913, Certain Information Relative To The Tuolumne, Stanislaus, Mokelumne, and Consumnes Rivers In California".

The major premise on which to base any reasoning as to the water available for a municipal supply from the Tuolumne River, based

on prior irrigation rights to the water, is the minimum seasonal water allowance necessary to irrigate the lands of the Modesto-Turlock irrigation districts. Several engineers familiar with irrigation requirements in California have placed this allowance at from 4 to 2½ acre-ft. per acre per year, net, that is, measured on the land to be irrigated. The larger figure, 4 acre-ft., was determined by Edwin Duryea, Jr., M. Am. Soc. C. E., from elaborate tests and measurements of water used over both districts, and is an average which he considers must be adopted as a safe minimum.* A figure of 2¾ acre-ft., net, has been given by Burton Smith, M. Am. Soc. C. E., formerly Engineer for the Turlock District, and the lower figure of 2½ acre-ft., net, has been mentioned by Messrs. Grunsky, Cory, and others, and is that adopted by Dr. Smith in his Senate report. Mr. Van Norden.

The board of Army Engineers, as shown in Mr. Wadsworth's report, have adopted a smaller water duty for irrigation, allowing 2½ acre-ft. per acre to be irrigated, measured at the point of diversion, and hence allowing nothing for evaporation and seepage in canals and ditches. In this difference lies the variance in the conclusions of Dr. Smith and Mr. Wadsworth. The report of Dr. Smith sums up as follows:

"The following calculation, based on the recent period of deficient run-off of Tuolumne River, indicates that for this period the Tuolumne River is not dependable, with 450 000 acre-ft. of storage for irrigation and 550 000 acre-ft. of storage for city water supply, for more than 230 000 000 gal. per day."

Here follows an elaborate calculation of the actual quantities of storage, in place and as the result of draft by irrigation and by the municipality, beginning with July, 1911, when it is assumed that all storage reservoirs would have been full. An equation quantity, x, is used to denote the monthly draft for city use. Evaporation during the late summer months from the reservoirs devoted to city supply is placed at 25 000 acre-ft. This calculation concludes with the following sentences:

"The condition of storage in the City reservoirs on January 1st, 1914, would be represented by 712 592 — 90 000 (accrued evaporation) — $29x$." "If we assume that the reservoirs are empty on that date, then we have the equation, $712\,592 - 90\,000 - 29x = 0$." "Then $x = 21\,469$, the amount of water, in acre-feet per month, which the city might have drawn during the period from August 1st, 1911, to January 1st, 1914." "This amount is equivalent to 233 000 000 gal. per day draft over the entire period of 29 months."

There were only 200 copies of this document printed, and most of these were distributed in the Senate, a copy being placed on the

* "A Study of Irrigation Heads in the Modesto and Turlock Irrigation Districts, California." *Engineering News*, September 11th, 1913, p. 502.

Mr. Van Norden. desk of every Senator. The fact that very few people have ever seen this report, and that its mention has been consistently avoided by newspapers and by those who have been actively interested in the furtherance of the Hetch Hetchy project to the exclusion of the consideration of all other plans, is interesting.

Under the present condition of affairs, the writer is of the conviction that the City of San Francisco cannot succeed in developing its future supply of water from the Tuolumne water-shed, or from any other undeveloped water-shed; but with the information and experience now at hand, let the city engineer begin anew, with the broad purpose of exploring every available source and method of supply, equally and fairly, without fear or favor, forgetting for the moment the money already spent and the rights acquired for developing the Hetch Hetchy. A solution satisfactory to the majority can then undoubtedly be found.

Mr. Smith. BURTON SMITH,* M. AM. SOC. C. E. (by letter).—On page 2180† the author assumes that 1 500 000 acre-ft. of storage can be provided, in about forty reservoirs scattered over the Tuolumne water-shed, at a cost of $30 per acre-ft.; and that the entire storage system can be operated with an 80% over-all efficiency. On these assumptions the basic estimates of yield and cost of the Hetch Hetchy project are deduced. Conclusions thus reached are nothing less than dangerous.

The writer, until recently and for a number of years, has been Chief Engineer of the Turlock Irrigation District, and has thus had occasion to examine most carefully and critically all storage possibilities in the entire Tuolumne water-shed—doubtless more exhaustively than any one else has done. Stated briefly, the results of these investigations are that, at most, 1 000 000 acre-ft. of at all practicable storage possibilities exist, and that the average cost of any such quantity would exceed considerably $40 per acre-ft.

These figures generally agree with those of the United States Geological Survey, as given by the Director, Dr. George Otis Smith, in his report on this subject made in accordance with the Senate Resolution of October 31st, 1913.‡

On page 2183† the author makes some quantity estimates which are based on 1 500 000 acre-ft. of storage—which, as just stated, is far too great—and various irrigation water requirements. Dr. Smith, in the report just mentioned, after stating that the duty of water in that locality is not yet fixed, assumes it at 2½ acre-ft. per annum for the gross acreage measured at the point of diversion from the river. This is undoubtedly lower than (in accordance with terms of the

* Oakdale, Cal.

† *Proceedings*, Am. Soc. C. E., for September, 1914.

‡ Senate Doc. No. 246, 63d Cong., 1st Sess., November 29th, 1913.

Raker Bill) the Secretary of the Interior of several years hence will determine as the water needs of the land. Fortunately for all parties concerned, it will then be a matter of supreme indifference what the city authorities or the farmers think now or will think then on this point. The only thing of any interest will be the opinion of the Secretary of the Interior of that time, based on experience. Nevertheless, each can now entertain himself by speculating on what that Secretary's decision will be; and the writer's judgment is that, at the very least, it will be 3 and more probably $3\frac{1}{2}$ acre-ft. per acre. Mr. Smith.

The writer would also be more than deeply interested in a system of operating the storage of such a quantity and character with an efficiency of 80%, or anything near it.

In short, the author's estimates in these particulars are quite too liberal.

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COMPUTING RUN-OFF FROM RAINFALL AND OTHER PHYSICAL DATA

Discussion.*

By Robert E. Horton, M. Am. Soc. C. E.

Robert E. Horton,† M. Am. Soc. C. E. (by letter).—Recognizing the fact that run-off is the result of many factors besides rainfall, the writer has for many years avoided its estimation from rainfall alone where possible. As far as he has gone, he has reached the following conclusions: Mr. Horton.

1.—Under the present state of the art, fairly accurate estimates of total annual run-off can be made in most cases from data that are generally available, namely, rainfall, temperature, and the physical characteristics and cultural conditions for the drainage basin.

2.—From the data usually available, it is much more difficult to make accurate direct calculation of the monthly or daily run-off for individual months than of the yearly average.

3.—From existing gaugings of streams, the percentage distribution of the run-off throughout different months, or other subdivisions of the year, can be determined approximately for different classes of streams and for different localities.

4.—Having given the total annual run-off and the law of monthly distribution, a good and fairly correct estimate of run-off, month by month, can be obtained.

This method of determining the relative monthly distribution of run-off has been designated the "distributive method", by the writer, to distinguish it from others by which an effort is made to ascertain the regimen of a stream, without actual gaugings.

It is a fact that the total run-off from a drainage area can be estimated with greater ease and accuracy than the run-off for the

* Discussion of the paper by Adolph F. Meyer, M. Am. Soc. C. E., continued from May, 1915, *Proceedings*.

† Albany, N. Y.

Mr. Horton. individual months. This is due to the fact that many elements, not taken directly into account in such calculations, are mutually counterbalanced in a large measure in the course of the annual cycle. This fact, that the annual run-off can be calculated with greater accuracy than the individual monthly run-off, led the writer, some years ago, to use for the latter purpose what may be described as a "distributive method". Mr. Meyer follows a somewhat similar procedure.

The writer has for years firmly believed that much better estimates of run-off could be made than have generally prevailed heretofore, and that such estimates should depend, not only on rainfall, but on other physical data pertaining to the drainage basin. Such procedure may be described as a "hydro-physical method" as distinguished from methods dependent on rainfall as the only or chief factor. Mr. Meyer's method involves ingenious ways of taking into account approximately the more important factors affecting run-off. Like all such methods, it is subject to revision, refinement, and elaboration, but the writer believes that some more or less similar hydro-physical method will shortly come into use among engineers for the solution of the extremely important problem of estimating run-off in the absence of actual gaugings. The author's discussion of the exceedingly complex phenomena involved in the hydrologic cycle is unusually lucid and accurate, although necessarily quite incomplete, and some data are quoted which are subject to misinterpretation.

Experiments by Transeau are cited which appear to indicate greater evaporation from a bare gravel slide than from any other condition of the soil. In these experiments there was a saturated medium exposed freely to evaporation all the time. Actually, the evaporation opportunity for the gravel slide is of short duration, and in some cases is continuous for the other soil conditions cited. The rate may be greater, but the total evaporation is less for the gravel slide than for the other conditions.

Methods of estimating run-off in the absence of actual gaugings are not uncommon, and, the writer believes, are a disgrace to the Profession. The only excuses which can be offered in some such cases are (1) ignorance, (2) laziness, (3) an intention to deceive. The first, the writer is thankful to say, is by far the most common. Hydrology is a new, vast, and rapidly growing science, and that the majority of engineers, busy mostly with other matters, should not be familiar with all the details of such a complicated subject is natural, and would be expected. We do not expect all engineers to be experts in subaqueous foundations, electric dynamo design, or isostacy. This criticism is not aimed at any one in particular, nor is it suggested by Mr. Meyer's paper or its discussion. It is given as the explanation of a condition.

Confining attention to methods of estimating year by year the annual run-off of a drainage basin in the absence of gaugings, we may classify as follows: Mr. Horton.

1.—The standard river method;
2.—The direct comparative method;
3.—The rainfall percentage method;
4.—The empirical method; and
5.—Hydro-physical or rainfall-loss methods.

The Standard River Method.—The standard river method consists essentially in adopting the results obtained for some stream which has been accurately gauged for a long period of years as applying generally without modification to any stream in the locality. This method is mentioned only to condemn it. In early days, when both stream gauging and run-off data were meager, and hydrologic principles not fully understood, this method gained wide acceptance, notably in the case of the use of the Croton and Sudbury River gaugings as a basis for estimating the yield of other streams under all sorts of conditions.* With present knowledge of the effect of different physiographic and cultural conditions on run-off, the adoption of the yield of any stream as applying to any other stream without a study of the comparative conditions of run-off for the two areas is unjustifiable, even in the absence of all data except those which can be obtained from an inspection of the water-shed.

In the absence of a good stream, well gauged, and having similar meteorologic and hydrologic conditions for the application of the comparative run-off method, and even as a good check on that method, the use of the rainfall-loss method is desirable.

The Direct Comparative Method.—In the direct comparative method, the measured run-off per square mile from a drainage area that has been gauged is assumed to apply directly, *pro rata,* to the area in question. This method is simple of application, and in many cases is no doubt as good as any, if not the best. Cases where its use is certainly legitimate arise where a stream has been gauged at one location, and its yield at another near-by location is to be determined. One of the two areas always includes the other. All the physical conditions are the same for both over a considerable proportion of the larger area, even though the physical conditions, rainfall, culture, etc., may be quite different in the part of the area not common to both. Even where the area of which the yield is to be determined comprises the whole or a part of the area that has been gauged, the direct comparative method may require correction to avoid serious errors. For

* As an illustration of recent advocation of this method, see article by W. L. Church, "Formulas and Computations for Horse-Power Value of Streams." *Engineering Record,* July 1st, 1905, pp. 11-12.

Mr. Horton. example: A stream has been gauged at a point where it emerges from the mountains, its drainage area being 100 sq. miles. Its yield is desired at a point on the plains below, where its drainage area is 150 sq. miles. The measured run-off for the upper area is, say, 30 in. By the direct comparative method, this would be assumed as the run-off rate for the entire area; but, suppose that, owing to differences in rainfall, slope, and cultural conditions, the actual run-off rate for the 50 intermediate square miles was only 20 in., then the true run-off for the 150 sq. miles would be $\frac{2}{3} \times 30 + \frac{1}{3} \times 20 = 26.67$ in.

The Rainfall Percentage Method.—In using the rainfall percentage method, the percentage of run-off to rainfall is determined from actual gaugings for some area or areas for which the rainfall and all the physical conditions are assumed to be the same as for the area for which the run-off is to be estimated. This method has a weakness in common with the direct comparative method, in that, when it has been applied heretofore, it has been the exception rather than the rule that the engineer applying it really knew that the physical conditions affecting run-off (other than rainfall) were the same or even essentially the same for the two areas. If he had known these facts in sufficient detail, he should have applied a correction (if any was needed) to the run-off of the intermediate area in using the direct comparative method, and he would not have applied the rainfall percentage method at all, unless for a crude, rough approximation.

The rainfall percentage method has another inherent weakness, in that run-off is a residue from rainfall, not a ratio to rainfall. It is what is left after certain natural losses have been deducted. Some of these losses, such as interception, are more or less proportional to the rainfall, and so far may be treated as ratios; other losses are either quite independent of rainfall, as evaporation and transpiration, or are dependent to an equal or greater degree on other factors. Soil evaporation comes in this class.

Run-off Formulas.—As to run-off formulas: Such of these as involve rainfall as the only factor belong in a class with the rainfall percentage method. They are nothing more than supposed aids to judgment in selecting the proper percentage (Grunsky's). Others take into account certain other factors, as slope (Justin's), and temperature (Vermeule's and Justin's).

Probably no simple formula can be derived which takes into account in a satisfactory manner all the factors of prime importance which affect annual run-off. Methods undoubtedly can be evolved which take all these factors into account.

The word "method" is here used in distinction from the word "formula", as indicating greater flexibility. In a "formula", the factors are fixed, though their values may vary. In a "method" of estimating run-off, the factors used may be different in different cases.

according to the data available, and yet the method retains its identity. Hydrology is largely a matter of methods. The guiding principle is that use should be made of all the available data, if the value of the result justifies the labor involved. Mr. Horton.

Rainfall-Loss Methods.—The method of estimating run-off by deducting the estimated rainfall losses from the measured rainfall is not new; it is the method customarily used in England, and is there often preferred to actual gaugings, the writer thinks without reason. The usual English application seems to the writer very crude; yet, considering the simple soil, cultural, and climatic conditions of the chalk and certain other regions from which water supplies are commonly derived in England, it becomes evident that the necessity for more detailed estimates is much less imperative than in the United States.

Larger drainage basins, less numerous rainfall records, and more complex soil and cultural conditions, are causes which serve to explain the less frequent application of this method in the United States as compared with England.

The late George W. Rafter, M. Am. Soc. C. E.—from whom, by the way, Mr. Justin got the suggestion for his own run-off formula—made some quite extensive applications of the rainfall-loss method, taking into account area and extent of different cultural conditions in the drainage basin and relative water losses for each. In 1904 the writer used a rainfall-loss method to explain the difference in the observed run-off of streams in the agricultural, forested, and barren deforested regions of Michigan. Mr. Rafter's and the writer's methods differ from one another, from the English, and from the author's, but the underlying principle of all is the same.

As developed in 1904, the writer subdivided rainfall losses into three parts:

1.—Interception;
2.—Transpiration; and
3.—Soil evaporation.

Mr. Justin criticizes Mr. Meyer's method on the ground that it does not take into account the slope of the drainage basin. This is not necessarily true—slope is one of the factors affecting run-off, but its effect is probably less than usually considered to be the case. The rainfall being correctly determined, slope may be considered as one of the influences affecting the water losses. Slope of the surface does not materially affect the interception loss. It has but small effect on the transpiration loss in humid climates where the normal ground-water supply is adequate for plant growth, except perhaps in the case of very steep slopes; because experience shows that crops grow as well on moderately rolling ground as on plateaus at the same

Mr. Horton. average elevation. In general, where infiltration exceeds plant requirements, the transpiration loss is little affected by slope of the surface. Slope does, however, materially affect the soil surface evaporation. For the same exposure to air and sun, the evaporation rate will not be affected by slope, but as the water does not stand on the ground as long on steep slopes as on flat lands, the duration of surface evaporation—or, as the writer prefers to call it, the "evaporation opportunity"—is reduced by increased slope. Soil evaporation, being the product of rate and duration, the total amount of this loss is directly affected by slope. Thus slope affects only one of the three principal sources of rainfall losses. Mr. Meyer's method includes an evaporation coefficient dependent on the physical characteristics of the drainage basin, and the effect of slope may very properly be included in selecting this coefficient.

Effective Slope.—Soil evaporation, as just explained, is affected by all the elements affecting water surface evaporation. In addition, it is influenced by:

1.—The quantity and distribution of rainfall;

a.—Reduction in rainfall reaching the ground through interception;

b.—Shading of the soil surface from sun and wind;

c.—Change in the absorptive and capillary character of the soil by tillage;

2.—Extent and density of vegetation;

3.—Slope of the ground surface;

4.—Depth and porosity of the soil; and

5.—Orientation.

It appears that soil evaporation is so complex that it cannot be reliably estimated in terms of temperature as the only variable.

Some experimental indication of the soil evaporation loss is obtainable from lysimeter experiments. Data of crop area, normal growth curves, and tables of duration of the growing season, afford a basis of estimating the amount of interception and soil shading, and experiments in forest meteorology furnish light on this phase of the subject. For the present, it appears to be necessary to include a coefficient or arbitrary factor, derived by judgment from a consideration of all these factors, in estimating soil surface evaporation.

Certain advantages of the hydro-physical method of estimating stream flow appear to be as follows:

1.—Run-off is treated, not as a ratio, but as a difference between water supply and water losses, which it is in nature.

2.—It takes into account a much larger number of the factors affecting stream flow than do other methods.

Mr. Horton.

3.—It is logical, in that it makes use of all the available information bearing on the subject.

4.—The selection of arbitrary factors or coefficients by judgment does not apply to the entire estimate, but is, or may be, limited to part of the factors, the other factors being based on direct observation or experiment.

Advocates of the rainfall-percentage method of estimating run-off, or of particular run-off formulas, will lose nothing by possessing themselves of the added knowledge of the drainage basin, needed in making an estimate of run-off by the hydro-physical method, and by so doing they may avoid committing gross blunders, such as have sometimes come to the writer's notice, where the direct comparison or rainfall-percentage methods have been used.

Transpiration is an exceedingly complex phenomenon. The writer does not think it is adequately covered or accounted for by Mr. Meyer's method.

The existing data of transpiration by crops are summarized in Briggs' and Shantz' papers, which, taken in conjunction with a census of crop production on the given area, seem to the writer to furnish the best possible basis for estimating this loss. Crop yield is not a precise measure of transpiration loss, but it seems to the writer that it is the best and simplest measure generally available. Transpiration and crop production both depend jointly on light, temperature, humidity, wind velocity, rainfall, and other factors. The yield of the crop is a measure of the resultant effect of all these factors on the vegetative activity of the plant. Transpiration is closely allied to the vegetative activity of the crop, and so the yield becomes a measure of this also. In using transpiration as a factor in stream-flow estimation, a number of the most complex and troublesome variables affecting stream flow are automatically taken into account.

Mr. Justin questions the possibility of securing data of transpiration losses independent of soil evaporation. He will do well to consult the results of the extensive experiments by Briggs and Shantz, J. W. Leather, and others, in which the transpiration loss for all sorts of common crops has been very accurately determined, independent of soil evaporation. The results are expressed in terms of the transpiration ratio or weight of water transpired per unit weight of dry grain or dry matter produced.

Fig. 19 must be considered as a highly generalized and only approximate average curve of transpiration.

The author's method divides water losses into two principal classes: "transpiration" and "evaporation". The writer prefers to use three: interception, transpiration, and soil evaporation. Interception varies with rainfall and with rainfall distribution. Its variation with tem-

Mr. Horton. perature and yield of crop, though probably appreciable, is relatively small, compared with the total, and may be considered as of the secondary order, and usually negligible for purposes of practical calculation. Interception is zero for zero rainfall, whereas transpiration and soil evaporation may be considerable for a month of no rain.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE DESIGN OF HYDRO-ELECTRIC POWER PLANTS

Discussion.*

By Messrs. H. Homberger and Arnold Pfau.

H. Homberger,† M. Am. Soc. C. E. (by letter).—This paper is to be welcomed from the point of view that it relates to the design of hydro-electric power plants in a comprehensive way, covering the entire field, and giving to the engineer who may be called on to do such work, a guidance in what manner to proceed. Mr. Homberger.

The situation which confronts the engineer who is in charge of designing a power-plant is very well expressed in the author's sentence: "This entire subject becomes one where the conditions surrounding each particular case must govern." In this very sentence is expressed the idea that it is almost futile to lay down fixed laws of design for hydro-electric power-plants. It is one of the characteristics of hydro-electric power work (a characteristic which makes it particularly interesting to the engineer), that it is impossible to generalize or standardize any part of it, and that it very rarely occurs that two problems are alike or nearly so. For this reason, specialists in hydro-electric work have considered it hopeless to write a paper, a treatise, or even a voluminous compendium in which any engineer put in charge of such work could find prescriptions or recipes, which he could pick out and use for the individual case which confronts him. More than in any other line, special preliminary study of each important case is necessary in order to obtain satisfactory results, and both the experience and the ingenuity of the engineer have a great deal to do with success or failure. Any engineer who has specialized in this field, and

* Discussion on the paper by J. D. Galloway, M. Am. Soc. C. E., continued from August, 1915, *Proceedings*.

† Mill Valley, Cal.

Mr. Homberger.

will let hydro-electric power plants built during the last 10 or 15 years pass in review before his mind's eye, will find a sufficient number of cases proving that the above-mentioned contention is justified.

After briefly mentioning the points of reference, Mr. Galloway begins his paper with his conclusions. Taking these up in order, comments on a few of the points will be offered.

The Load Factor.—It is pointed out that the load factor has a very important bearing on the design of the prospective power-plant. In a general way, this is probably correct. Load factors are stated to be from 15 to 25% on a lighting load, 50% on a street-car system, and from 80 to 90% on milling or mining. There are only few power

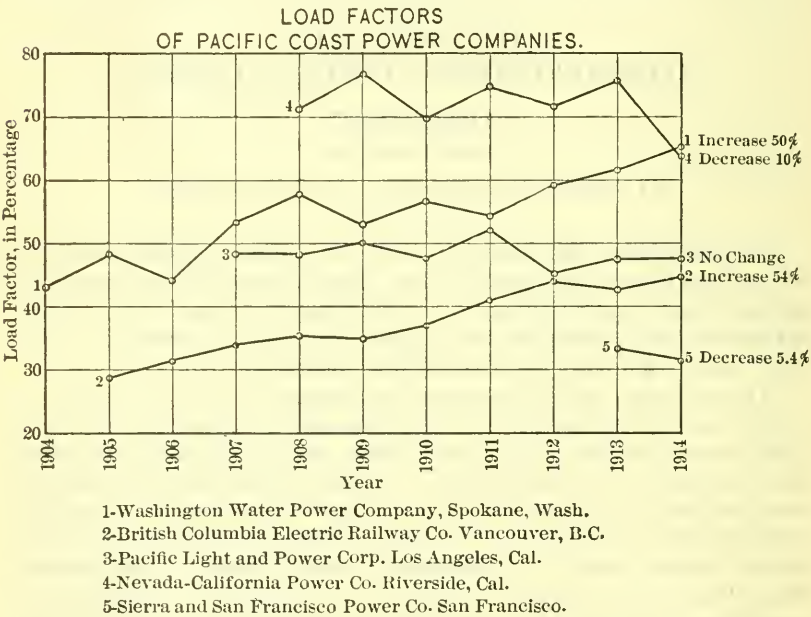

FIG. 12.

systems of medium or large size which can maintain such a well-defined load that they may be put under any of the above-mentioned classifications. It is the endeavor of every power company to obtain a mixed load and to improve the load factor, and the history of the development of the power business shows that often, after a few years, the load factor had become much different from what it had been expected to be when the plant was conceived and constructed. The diagram, Fig. 12, shows the development of the load factors of some of the larger Pacific Coast power companies. It may be seen that in general the tendency of the load factor is to improve, often due to the efforts of the soliciting and commercial departments of the power

companies to invite customers to purchase power at the time of day when the average load is lowest, and also to solicit power consumption in small factories and in homes, a class of business in which a few years ago power companies were less interested. Mr. Homberger.

The load factor of a railway system is variable, and depends largely on the size of the railway system and on the operating conditions. It may be very much higher than 50%, and also considerably lower. Mr. Galloway gives the load factor of the Sierra and San Francisco Power Company as 50 per cent. The principal part of the business of this company is to furnish power for the street cars of San Francisco, with practically no suburban lines. The load factor in 1913 and 1914 was about 30 per cent.

As time goes on, the load of the power company may change entirely in character. Some power companies were established principally to furnish power to mining districts. Mines generally work in three 8-hour shifts all the year around, and consequently furnish a load factor which is very high; but mines, as a rule have a somewhat limited life. Power companies must plan for a much longer period. In accordance with laws which have been made in the United States during the last 15 years, and others still in the making, one may assume that a power company is to last from 40 to 50 years. When the mining business begins to fail, the power company must look for other markets for its product, and may be very glad to enter into a strictly lighting market, with a load factor of 20%, in order to take off their hands a surplus which it had been marketing under an 80% load factor.

It may be seen from this that the load factor of a prospective power-plant cannot really be foreseen with any close exactness, and is largely guesswork. Unfortunately, this is not a problem of exact science, but is a variable quantity, depending on developments which cannot be anticipated.

The Rules of Economy.—The following comments are offered on the statement:

"In some cases where there is an abundance of water, it is not necessary to strive for the highest efficiency of the various parts, as the losses are not vital."

This is a theory which, unfortunately, has been followed in a number of cases in the design and construction of power-plants. Efficiency is of great importance under any circumstances. It seems to be immaterial how much water may be flowing in a river as long as the entire minimum flow is not required for the power development. It can be shown, however, that to select equipment of low efficiency, on the theory that the water has no value, will lead to the construction of a power-plant which is not economical in operation. Taking it for granted that, for a generating plant of a given size, the efficiency

Mr. Homberger. of the electric apparatus was fixed by the size and type of apparatus which is offered by manufacturers, let us say that, for a plant of 100 000 kw., 160 000 h.p. was to be developed on the shafts of the water-wheels. Water-wheel makers cannot standardize apparatus like manufacturers of electric machinery, and it is quite likely that the power company will receive proposals for the hydraulic prime movers which vary 10% in the average efficiency guaranteed for an output between 80 000 and 160 000 h.p. in the power-plant. Let us assume that the effective head in the plant was to be 1 000 ft.; that one design of the water-wheels guaranteed an efficiency of 80%, and another one 90 per cent. This would seem to indicate that for the plant the quantity of water which would have to be carried through the pipe lines, head-works, and canal system would vary 12½ per cent. Assuming that the effective head was to be the same for both types of prime movers, it would mean that a pipe of greater capacity would have to be provided for the machine of lower efficiency, and that all the work above the pipe line would have to be correspondingly enlarged. It can be shown that there is only an apparent economy, and that the selection of apparatus of lower efficiency will be very costly, on account of the heavier investment in plant construction. The writer remembers an engineer, who was engaged in the design of a power-plant, making the statement that he had selected turbines, not as much with consideration of their high efficiency, but principally their first cost. That the plant was to contain ultimately four units, at first there were to be two units, and, while the power business was building up and developing, the minimum flow would be twice the capacity of these two units, and that it was therefore not necessary to put in wheels of high efficiency. Later, when two additional units would be installed and the entire minimum flow would be utilized, machines of higher efficiency would be selected. This practically means extravagance during the development period of a new business, and economy after the business is established.

Penstock Pipe and Number of Units in a Plant.—Mr. Galloway gives some rules for the number of pipe lines and the number of units in a power-plant. In the layout of pipe lines, the writer cannot see any reason why there should be not less than two pipes; why, in high-head plants, one pipe may be used to serve two generating units even of larger size; why, for four units, two pipes are sufficient; nor why, in a low or medium-head plant, where the quantity of water is not large, each generator should have its own pipe. All these points must be determined by economy of construction and safety of operation. There may be conditions where a high-head plant operates four units from one pipe, though in other cases it may be preferable to equip each individual unit with its own pipe. In many cases it has been economical to use single pipes for a number of units in the upper part

Mr. Homberger.

of the profile, where the pressures are low and a large pipe can be constructed of light material, and to subdivide the line into two or more single lines as the pressure increases. In general, the writer would refer, on this point, also the question of number of units in the plant, to Mr. Galloway's own statement, that the conditions surrounding each particular case must govern.

Regarding the number of units in a plant, the determining factors are: first, the maximum economical size of generating unit that may be constructed; secondly, the factor of safety required for continuous operation of the system; thirdly, whether there are any other hydraulic or steam stand-by plants to help out, making it permissible to reduce the factor of safety of operation in the plant to be designed; and fourthly, the character of the load on the power-plant. The author's conclusion, that the number of power units in a small station should be at least three. and in any station not more than four or five, unless the total available power is greater than can be generated by five units of maximum size, does not appear to be borne out by power-plant practice, either in the United States or abroad.

The Limitations of Impulse-Wheels and Turbines.—The definite relation between pitch diameter of runner and maximum jet diameter has been referred to, and 15 has been given as the minimum for the ratio between the two. Aside from the efficiency, which is affected by this ratio, it may be mentioned that the lower limit is fixed by problems of design. Assuming that the buckets of an impulse-wheel are to be fastened singly to the wheel proper, and the fastening of the buckets is accomplished by bolts traversing bucket and wheel in a direction parallel with the shaft of the machine, it will be readily seen that the necessary bolting space governs the largest size of bucket that may be placed on a wheel of given pitch diameter. The strains on the bolts are dependent on the head under which a wheel of a given size of jet operates; consequently the working head has a direct influence on the lower limit of the ratio and makes it variable. If 15 may be the limit for a head of 2 000 ft., 12 may be satisfactory for a 1 000-ft. head, while, with what is termed a low head on the Pacific Coast, the ratio in extreme cases may be even reduced to 9. The interlocking-chain type of bucket has made it possible to use these low ratios for much higher heads than was possible with the old type of bucket fastening, where each bucket requires two individual bolts. Ratio and specific speed have a constant relationship for all heads, and the following tabulation covers the cases generally occurring with main generating units:

Diameter of jet	1	1	1	1	1	1	1	1	1	1	1	1
Pitch diameter of wheel...	9	10	11	12	13	14	15	16	17	18	19	20
Specific speed.	5.88	5.31	4.77	4.4	4.05	3.76	3.5	3.3	3.1	2.92	2.77	2.64

Mr. Homberger. For exciter units, direct-connected to impulse-wheels, the ratio between wheel diameter and jet diameter often reaches much higher values, so that the specific speed may go even below one.

For Francis turbines, the following tabulation gives a range of specific speeds, where R designates the ratios between the peripheral velocity of the runner and the spouting velocity, as characteristic for runners of different types:

R.............	0.585	0.625	0.665	0.7	0.75	0.85
Specific speed...	13.55	20.3	29.4	40.7	56.1	74.8

Both limits of this tabulation have been exceeded in practice; the lowest specific speed the writer knows of is about 10, the highest about 85. Such extremes, however, can only be resorted to with a sacrifice of efficiency, and the very low specific speeds also result in the disadvantage that the runners are bound to wear more rapidly than they do at higher speeds. In order to meet the standard speeds of generators of not too large dimensions, say from 300 to 600 rev. per min., it is necessary to select turbines of high specific speed for low heads, and *vice versa*. A runner of low specific speed becomes large in diameter, and since the area between the runner vanes is proportional to the quantity of water that passes through the turbine, the width of the runner is reduced with increasing diameter. The narrower the passages, the higher the frictional losses, and, at the same time, the wear increases, particularly if the water carries silt. In such cases a higher speed should be selected, even if the generator becomes more expensive on account of special construction, or an impulse-wheel should be resorted to. If the impulse-wheel with single jet cannot furnish the necessary power, two or even more jets may be applied to each wheel, or more than two wheels may be adopted for one unit. The efficiency of such machines is somewhat lower than that of single-jet wheels, but often this is a lesser evil than a Francis turbine, which requires frequent replacement of runners.

In general, it may be said that, for heads ranging from 300 to 600 ft., with specified capacities of the units, the specific speed gives a guidance whether a Francis turbine or an impulse-wheel should be selected. The former will be preferable when the specific speed is greater than 15, the latter when it is less than 5. Between these limits special construction of impulse wheels with multiple runners and multiple jets will generally give more satisfactory results in service than Francis turbines, particularly when high head and water not free from silt prevail.

Mr. Pfau. ARNOLD PFAU,* ESQ. (by letter).—In the introductory of this able paper, Mr. Galloway remarks that very often little consideration of the relation of the machine to other parts of the plant is given, etc. If

* Milwaukee, Wis.

Mr. Pfau.

this fact is admitted by a practical engineer engaged in the design of complete plants, what shall we expect of the practice of builders of so-called "home-made" plants. Too much stress cannot be laid on the importance of this very subject, and the writer believes that it is the duty of the engineer, and also to the mutual benefit of the engineer, the manufacturer, and the purchaser, or of the financing interests behind them, to emphasize the importance of proper co-operation of the various parties engaged in the development of water-power with a view of obtaining such results as will guarantee the lasting success of the whole enterprise.

Mr. Galloway has touched on a subject which opens a wide field for discussion, and much remains to be done to raise the efficiency, not only from the engineering, but also (and probably more so) from an ethical, point of view.

There can be no doubt that true co-operation means efficiency.

The true co-operation of engineers or engineering concerns among themselves may be found to constitute a powerful weapon against the condemnable practice of parties lacking either technical abilities or conscientiousness. It may also serve to reduce the practice of "home-made building" of plants by parties not fit to attempt such work themselves, who, however, may have been justly forced to such a procedure on account of disappointments experienced before, with engineers lacking professional ability or scrupulousness.

The true co-operation of the manufacturers among themselves would serve to prevent abuse of their confidence on the part of prospective purchasers or engineers. Such abuse exists in cases where manufacturers are requested to furnish elaborate plans, specifications, and guaranties, when these are only intended for use in preparing a report on a proposition which may be justly called nothing more than a "wild-cat scheme." Is it not "abuse" when leading manufacturers are requested to expose their engineering and prepare plans and specifications which will be used by the purchaser as a basis for final specifications which are then sent to manufacturers who themselves are incapable of designing what has been collected by the engineer and has been combined as the best solution of the problem? Is it not an injustice to the conscientious client to make him carry the manufacturer's burden of the general expenses caused by parties who are neither connected with him nor friendly toward his purveyor? Co-operation, also, along the lines indicated, would have as beneficent an effect as co-operation in regard to technical matters. Space does not permit one to go any further here. If the remarks made serve to stir up the sentiment, they have answered the purpose for which they were brought forth.

Mr. Galloway states that the turbine designer may set forth requirements which the generator builder cannot meet. The writer's experi-

Mr. Pfau. ence leads him to believe that it more frequently happens just the other way; that is, that the generator builder has fixed conditions which, normally, cannot be met by the turbine builder. It is only a few years ago that the practice existed first to look for a standard generator which could be obtained at a favorable price—if not second-hand—and then force the turbine manufacturer to build a machine which would meet the sometimes very freakish requirements of power and speed. Even now a generator is often purchased with a fly-wheel effect of its rotor insufficient to secure satisfactory speed regulation of the unit, although, with little extra expense, such fly-wheel effect could be most efficiently built into the rotor without impairing the efficiency of the generator.

A hydraulic prime mover, like no other prime mover, is dependent on the manifold characteristics of its operating fluid supplied by Nature. It is the head and flow which are fixed with each development; it is the quality of the operating water, the climate, and many other factors which enter into consideration in the selection of the proper size and type of turbine; and it is natural that the specialist should know most about it, and therefore he should be given all the underlying data for deciding on the best selection.

Under the heading "Load Factor" Mr. Galloway suggests operating the steam plant over the peak and raising the load factor on the hydro-electric plant. The writer invites attention to one phase of this condition: Suppose it costs more to produce a steam, or other than a water-power, kilowatt, and suppose the water-power is combined with a storage reservoir; then it would seem to be more economical to let the water-power plant take care of the load fluctuation and set the governors of the steam plant so that the steam-power generating unit will operate at the point of highest economy. This can be done by providing a load-limiting device, and a relay on the governors of the steam-power units, which is set so that it does not allow the power to be changed beyond the best economy point in either direction, except in case the load demand should (through short circuit or other disturbance) become less than the power produced by the steam (or other prime mover) unit. Consider, for instance, an electric-railway system operated jointly by steam and storage water-power prime movers. The load factor of such a system may be very low; however, all the stand-by capacity must be ready at a moment's notice to take care of sudden peaks. It would certainly be most economical to let the steam plants always carry the minimum load, so that these units would be prevented from operating at a point of commercially low economy.

A splendid and welcome remark has been made by the author under the heading "The Value of Power", namely: that if money is to be spent, it is much better to apply it at the receiving end. It is not because the writer belongs to the manufacturing group that he points

to the importance of the remark "at the receiving end", but more because, in his experience, he can cite many cases where the financial success of an enterprise has been seriously hampered by neglect of this very question. Mr. Pfau.

Thousands and hundreds of thousands of dollars are spent on hydro-electric developments. The most splendid engineering and construction work is carried out, and nothing is left undone to render the construction work perfect. How different is sometimes the procedure in the selection and purchase of the machinery; how little money is left for the prime mover, the most vital factor of the whole enterprise, and the one which converts the natural resources of power into the "dollars and cents" which are intended to flow into the pockets of the owners. It is either the lack of adequate financial resources, or too extravagant construction work, or the lack of appreciation of the importance of properly selecting and purchasing the prime-mover equipment that creates so many hydro-electric plants in which the prime mover constitutes the weakest and least dependable link of the whole system. To illustrate this statement the writer will cite a remark made by the operating manager of one of the larger hydraulic plants in the United States: "If we had known before what we know now, we could have accepted the highest bid, paid the bidder extra for nickel plating every nut on his equipment, and still be money ahead, because we would not now have the continual expense of up-keep and replacement". The truth of this statement is acknowledged when one looks at the graveyard of parts out of commission stored near his power-house.

Under the heading "Generators" Mr. Galloway cites the possible numbers of revolutions per minute for 60-cycle generators. The speed of 150 rev. per min. may be considered low for medium heads. Single-runner, vertical-shaft units operating under low head, however, may operate under considerably lower speeds, and, therefore, synchronous speeds of 120, 100, 90, 80, 72, 60, and 57½ may be justly added. In the early Nineties a low-head plant was built in Switzerland with vertical-shaft, Jonval turbines, directly connected to alternating-current generators at 28 rev. per min. These generators, however, did not satisfy, and were replaced by horizontal-shaft generators, of double capacity, operating at 56 rev. per min., and had beveled mortice gears, driven from the vertical shafts of two turbines each. It should be borne in mind that low-speed generators have a lower efficiency than high-speed machines of equal capacity. This disadvantage, however, may be more than offset by the increased efficiency of a single runner of large diameter over that of a plurality of small runners, and it may also be offset by advantages of simplicity of design and better maintenance of a single, vertical-shaft arrangement with outside (accessible) gate rigging, etc.

Mr. Pfau. A more detailed discussion of this subject may be found in the writer's paper on Topic No. 20, to be presented before the International Engineering Congress, 1915, at San Francisco.*

Under the heading "Limitations of Impulse Water-Wheels", Mr. Galloway states that experience shows that the ratio of wheel diameter to jet diameter should not be less than 15, in order to obtain the best efficiency. (Reference is also made to the term "pitch diameter", but the writer would suggest the adoption of the term "impact diameter", being the diameter of the wheel circle tangent to the axis of the jet.) The ratio given represents a very safe figure. There are many factors, of course, which determine the best efficiency. Some eight years ago this was a subject of acute discussion among engineers connected with the design of buckets, but now it has been generally acknowledged that the angles and the number of buckets greatly influence the question of best efficiency. The term "specific speed" is applicable to impulse-wheels as well as reaction, or other, turbines. It explicitly represents nothing else than a ratio, as follows:

For impulse-wheels............ $N_s = C \ \frac{d}{D}$;

For turbines.................. $N_s = C^1 \sqrt{\frac{B}{D}}$;

C and C^1 being constants of fixed value for a fixed design only.

The explicit value of C is:

$$C = \frac{60 \times 2\ g \sqrt{2\ g} \sqrt{\frac{\pi}{4}} \sqrt{62.5}}{\pi \sqrt{550}} \times U^x \times \alpha \times \sqrt{e_j\ e_b}$$

in which U^x = peripheral speed coefficient at impact diameter;

$$U^x = \frac{D\ \pi \times (\text{rev. per min.})}{60 \sqrt{2\ g\ H}};$$

$\alpha \doteq$ the coefficient of velocity of the jet;

$$\alpha = \frac{Q \times 4 \times 144}{\pi\ d^2 \sqrt{2\ g\ H}}, \text{ if } d \text{ is in inches};$$

e_j and e_b = the efficiency of the jet and the buckets, respectively; and α and e_j are related values;

$$e_j = \frac{M}{2} \left(\alpha \sqrt{2\ g\ H}\right)^2$$

$$\frac{M}{2} \left(1 \sqrt{2\ g\ H}\right)^2 = \alpha^2.$$

From the foregoing it can be seen that the specific speed is a function, not only of the ratio of jet and impact diameter, but also of

* As this paper has not yet appeared, it has been found inopportune to publish some of the data here.

Mr. Pfau.

the peripheral speed of the wheel, the efficiency of the jet, and of the buckets, the latter being influenced naturally by the discharge losses, consequently by the relative discharge angles of the buckets. It thus follows that considerable flexibility of the characteristic can also be obtained with an impulse-wheel by properly selecting its fundamental values embodied in the design.

A discussion along these lines was first opened in connection with the design of the impulse-wheels of the Kern River Plant No. 1, of the Southern California Edison Company.* The correctness of the design is proved, inasmuch as these buckets are still in operation and show absolutely no signs of corrosion, in spite of the fact that heavy duty is imposed on them due to the continuous partial impingement of the jet issuing from a governor-controlled deflecting nozzle.

The values of specific speed applied to turbines are still more flexible than those applied to impulse-wheels, and therefore a very wide range is found in practice. With turbines, the term "specific speed" also represents the product of a constant multiplied by a ratio. The ratio is in some respects similar to that of the impulse-wheels, inasmuch as for the jet diameter is partly substituted the clear height of the runner. The value of the constant, C^1, is explicitly:

$$C^1 = \frac{60 \times 2\,g \sqrt{2\,g} \sqrt{\pi} \sqrt{62.5}\; U^x \sqrt{V_3^{\,x}} \sqrt{e}}{\pi \sqrt{550}}$$

in which U^x represents the peripheral speed coefficient of the runner at the entrance; $V_3^{\,x}$ is a radial velocity component, or that velocity coefficient which is figured from the formula:

$$V_3^{\,x} = \frac{Q}{D\,\pi\,B \sqrt{2\,g\,H}};$$

Q being the discharge, in cubic feet per second, D the average entrance diameter, and B the vertical height of the runner. e is the total efficiency of the turbine. As the values, U^x and $V_3^{\,x}$, vary throughout a very wide range, it is evident that, for a fixed efficiency, e, the specific speed of a turbine also may vary throughout a very wide range. Another low specific speed of a successful turbine is 12.8 (English) or 56.8 (metric). It is a replacement of a Girard (or action) turbine by a Francis (or reaction) turbine, whereby head, power, and speed were fixed by the conditions of the existing unit.

The highest specific speeds are fixed by the progress of the art in designing high-speed, high-power runners, although they are applicable only to low heads. Specific speeds exceeding the value, 100, are in successful commercial operation. Small model runners have

* The diagrams were published (although on a rather distorted scale) in *Engineering News*, December 24th, 1908.

Mr. Pfau. been tested with specific speeds as high as 207 (English) or 920 (metric), and with very satisfactory efficiencies.

A curve has been given in the paper previously mentioned showing the specific speed as a function of the head, from which it may be determined, and an empirical formula will also be found from which specific speed can be computed for a given head, or *vice versa.*

Mr. Galloway correctly states that trouble is invited with the adoption of a specific speed too low for high-head Francis turbines; such runners have a tendency to operate as non-ventilated, action turbines. The failure of admitting air under such conditions is responsible for the corrosion of both runner and guide-case, and in many cases also for vibrations and hydraulic noises.

One more discussion may be in order in connection with head, in the selection of a water-wheel, and particularly with regard to the question of utilizing the draft-head. Draft-tubes, however, have been applied also to impulse-wheels, with more or less success. If correctly applied (which can only be done in connection with a float controlling an air-valve by which the level of the discharge water is kept within a certain distance of the lowest point of the wheel buckets), negative pressure is formed in the wheel housing, according to the height of the suction column. This negative pressure accelerates the jet issuing from the nozzle, and thus virtually increases the effective head accordingly. It can be readily seen, however, that the velocity of the water leaving the buckets is not reduced, and consequently its energy is not recovered.

Not so with the draft-tube applied to a reaction turbine. There it is not only the additional suction head that is utilized by its application, but additional energy is obtained by the retardation of the water column discharging from the runner. This is particularly important in connection with low heads, where the suction head may constitute a large percentage of the total available net head, and more so with high specific speeds, where the velocity of the water leaving the runner is relatively high ($0.45 \sqrt{2 g H}$ or more), which would seriously impair the efficiency of a turbine, if not carefully reduced by a properly designed draft-tube.

The writer can cite a case where the output of a standard stock turbine was increased 34% by no other change than that from a short, stub draft-tube to a concrete draft-tube of proper design. It is advisable, therefore, with low-head developments, to hold the turbine manufacturer responsible for the dimensions of the draft-tube.

Mr. Galloway's paper should receive the best of study, and the many points mentioned by him should be used liberally for discussion and further explanation. This would serve to unite better the parties connected with the development of water-power, from engineering, as well as social or ethical, points of view.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE PUMPING PLANT OF THE MORENCI WATER COMPANY

Discussion.*

By Lindsay Duncan, M. Am. Soc. C. E.

Lindsay Duncan,† M. Am. Soc. C. E. (by letter).—Although the milling of low-grade ore requires from 5 to 10 tons of water per ton of ore, the water supply is seldom given sufficient consideration in the location of ore mills. Mr. Duncan.

One would infer that at Morenci the mills and town struggled along for some years with an inadequate supply of very poor water, until they finally combined their resources and built the very complete plant which Mr. Du Moulin has described.

Off-hand, it would seem to the writer that a supply of clear water for the pumps could have been obtained readily by driving a filtration gallery under the river bed. This gallery could have started from the shaft, which would have been available as a suction sump. The writer supplemented the water supply of the Nevada Consolidated Copper Company, at McGill, Nev., in this manner, by driving a tunnel 1 150 ft. across the valley of Duck Creek, and obtained a flow of 1 500 gal. per min. from a depth of 50 ft. The overburden was composed of alternate layers of clay and gravel, and these were broken up by removing the lagging from the roof of the tunnel at 50-ft. intervals and starting "runs" which caved the ground up to the surface.

The low vacuum reported seems to the writer to be due to air leakage into the condenser. Minute openings in the cast-iron condenser shell, or imperfect joints in the exhaust piping, will affect very seriously the vacuum of a condensing engine.

* Discussion of the paper by W. L. Du Moulin, Assoc. M. Am. Soc. C. E., continued from August, 1915, *Proceedings*.

† McGill, Nev.

Mr. Duncan.

The writer also believes it to be bad practice to introduce an oil separator between the exhaust nozzle and the condenser, as it is bound to reduce the vacuum available at the engine. It would be better to filter the condensate and put in a surface blow-off, or skimmers, in the boilers.

In the absence of any statement, the writer assumes that the vacuum was measured at the exhaust nozzle, where the deleterious effect of the oil separator would be felt.

Considering the low vacuum, the duty obtained by these engines is truly remarkable, and the writer would be interested in additional details of the tests. A statement of the number of gallons of water pumped per pound of oil burned, together with the characteristics of the oil and the method of measuring the water pumped, would round out the paper, particularly if the author were able to give the duty per pound of oil of the new plant under test conditions and also after it had been run for several years. The writer's experience with high-grade Corliss engines leads him to believe that a 30% increase in steam consumption is to be looked for in a plant 10 years old.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE TWELFTH STREET TRAFFICWAY VIADUCT, KANSAS CITY, MISSOURI

Discussion.*

By Messrs. F. W. Green, L. J. Mensch, Howard W. Holmes, and M. M. Upson.

F. W. Green,† Assoc. M. Am. Soc. C. E. (by letter).—The author has made a noteworthy contribution to engineering literature. Unique seems to be the word best adapted, adjectively, to apply to a structure which is neither straight as to alignment, level as to gradient, symmetrical as to transverse loading, nor orthodox as to the tenet of design which requires that indeterminate stresses must be solved by being avoided. Mr. Green.

Moreover, it is unique in that the embellishments of art have been invoked, with very gratifying success, to commend to the esthetic emotions a structure primarily intended to be useful. This, too, under circumstances most unpropitious—an ugly *situs,* an unbalanced subject, and an inauspicious environment; and, as the author states, "without any extra funds for adornment".

A consideration of the problem creates a feeling of dismay, which is superseded later by one of admiration for the ingenuity shown and the resource developed in mastering the many baffling features. To the mind mathematically inclined, the graphical method for solving the integrals shown on pages 1029 and 1030‡ will commend itself both for elegance and simplicity. The method for obtaining moments and deflections for the rectangular frames (given on page 1031‡) by the solution of ten simultaneous equations, though no laughing matter,

* This discussion (of the paper by E. E. Howard, M. Am. Soc. C. E., published in May, 1915, *Proceedings,* and presented at the meeting of September 1st, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Stamps, Ark.

‡ *Proceedings,* Am. Soc. C. E., for May, 1915.

Mr. Green. offers opportunity for mirth: "When a structure is so formed that the stresses in it cannot be determined by statics, it is said to be a redundant system; and the principle of least work applicable to it is as follows, etc."* Do we not have here a paradox? Is it strictly proper to say that the solution of these ten simultaneous equations involves the principle of least work? The author says: "By arranging these * * * in tabular form, the solution is found with comparatively little labor." Did he intend "little" in the sense of *"multum in parvo"*?

Seriously speaking, however, the impression uppermost in the writer's mind, after reading this paper, is that this work marks another important instance of the present tendency in American development to pass from the stage of pioneer civilization, with its temporary, but economical and expedient, types of buildings, works, ways, and systems, to an era of permanence, beauty of line and mass, sufficiency, and efficiency.

The design of this structure is a bold departure in many respects from the established canons of design, and the novice should be admonished not to undertake liberties of this kind unless and until he is absolutely sure of his ground. For example, a comparatively slight settlement of the foundation under one of the 120-ft. columns conceivably might have disastrous consequences.

Mr. Mensch. L. J. MENSCH,† M. AM. SOC. C. E. (by letter).—Reinforced concrete seems to enlist the brightest engineers under its colors. Though viaducts like that described in this paper have often been built of steel, one would search American literature in vain for a description of such viaducts in which the attempt was made to design the structure for temperature and other so-called secondary stresses, as was done for the Twelfth Street Trafficway Viaduct of Kansas City. Because such refined—but often very necessary—computations are rare in American practice, we need not be surprised that, in one of the first attempts, such computations lack the simplicity, elegance, and surety of the schooled designer. Not until a great many of our students take post-graduate courses in the design of so-called "higher structures" can a material change be expected in this matter. Take, for example, Fig. 2 and its explanation. The writer has designed indeterminate structures for more than twenty years, yet he can follow the reasoning only with great difficulty. There is no need for such lack of clearness.

The problem can be solved entirely, with much less effort, by finding the unknown conditions of the supports at A_1 and C (Fig. 26). A sliding joint was assumed at A_1, hence the only unknown that can act there is the vertical force, P_1. The point, C, is fixed, hence the support will exert on the column, A_0 C_1, a thrust, H, a pull, P_2, and a moment, M_c.

* Merriman's "Mechanics of Materials", 10th Ed., p. 321.

† Chicago, Ill.

These unknown reactions can easily be found by the rules for the change of angles and for the deflection of a beam: Mr. Mensch.

$$\varDelta\ \alpha = \int \frac{M ds}{E\ I} = \frac{1}{E\ I}\ \text{(area of diagram of moment)} \qquad (1)$$

$$\varDelta\ y = \int \frac{M x ds}{E\ I} = \frac{1}{E\ I}\ \text{(static moment of diagram of moment about the point, the deflection of which is sought)} \qquad (2)$$

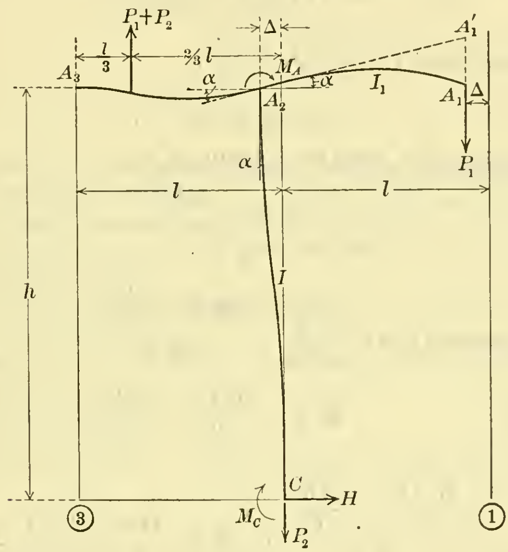

FIG. 26

For the force, P, acting at the end of a cantilever,

$$\varDelta\ \alpha = \int \frac{P x dx}{E\ I} = \frac{P l^2}{2 E I}$$

$$\varDelta\ y = \int \frac{P x \times x dx}{E\ I} = \frac{P l^3}{3\ E\ I},$$ provided I is constant.

$A'_1\ A_1$ can be considered the deflection of the point, A_1, due to P_1, or

$$l\ \alpha = \frac{P_1\ l^3}{3\ E\ I_1},$$

or

$$\alpha = \frac{P_1\ l^2}{3\ E\ I_1} \qquad (3)$$

It is true, as the author has stated, that the point of inflection of $D\ B$ is $\frac{1}{3}\ l$ from D, and, in order to preserve the equilibrium, we can

Mr. Mensch.

imagine that P_1 and P_2 are acting at this point, producing a moment, $(P_1 + P_2)\ \frac{l}{3}$, at A_3, and $(P_1 + P_2)\frac{2}{3}\ l$, at A_2.

Then
$$\alpha = \frac{(P_1 + P_2)\frac{2\ l}{3} \times \frac{2\ l}{3} \times \frac{1}{2} - (P_1 + P_2)\frac{l}{3} \times \frac{l}{3} \times \frac{1}{2}}{E\ I_1}$$

$$= (P_1 + P_2)\frac{l^2}{6\ E\ I_1} \qquad (4)$$

From Equations 3 and 4, $\frac{P_1 + P_2}{6} = \frac{P_1}{3}$,

or
$$P_1 = P_2 \qquad (5)$$

We can determine, also, from the column, $A_2\ C$,

$$\alpha = \frac{M_c\ h - \frac{H\ h^2}{2}}{E\ I} \qquad (6)$$

and from Equations 3 and 6, $\frac{P_1\ l^2}{3\ E\ I_1} = \frac{M_c\ h - \frac{H\ n^2}{2}}{E\ I}$

let $n = \frac{h}{l}\ \frac{I_1}{I}$,
$$M_c\ n - \frac{H\ h\ n}{2} = \frac{P_1\ l}{3} \qquad (7)$$

From $A_2\ C$,

$$\Delta = \frac{\frac{M_c\ h^2}{2} - \frac{H\ h^3}{6}}{E\ I}, \text{ or } M_c - \frac{H\ h}{3} = \frac{2\ E\ I}{h^2}\ \Delta \qquad (8)$$

By taking moments about A_2,

$$(P_1 + P_2)\frac{2}{3}\ l + P_1\ l + M_c = H\ h \qquad (9)$$

Equations 7, 8, and 9 have three unknowns, and we easily obtain

$$\left.\begin{aligned} H\ h &= \frac{2.4\ E\ I}{h^2}\ \Delta\ \frac{1 + 7\ n}{1 + 2.8\ n} \\ M_c &= \frac{2.4\ E\ I}{h^2}\ \Delta\ \frac{1 + \frac{7}{2}\ n}{1 + 2.8\ n} \\ P_1\ l &= 3.6\ E\ \frac{I}{h^2}\ \Delta\ \frac{n}{1 + 2.8\ n} \\ M_A &= 8.4\ E\ I \times \frac{\Delta}{h^3} \times \frac{n}{1 + 2.8\ n} \end{aligned}\right\} \qquad (10)$$

Mr. Mensch.

If the structure is cast monolithically, without any joints, we can place $I = \frac{b \times d^3}{12}$.

The moment of resistance $= \frac{b \times d^2}{6}$, and

$$\text{the stresses at } C = \frac{M_c}{\frac{b \times d^2}{6}} = \frac{2.4\, E \frac{b \times d^3}{12}}{\frac{b \times d^2}{6}} \frac{\varDelta}{h^2} \frac{1 + \frac{7}{2} n}{1 + 2.8\, n}$$

$$= 1.2\, E\, d\, \frac{\varDelta}{h^2} \frac{1 + \frac{7}{2} n}{1 + 2.8\, n} \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

The moment of inertia of the columns varies, and, assuming the section of the base only to govern, $n = 4.46$; of the top only to govern, $n = 2.76$, which produces hardly any difference in the value of the fraction, $\frac{1 + \frac{7}{2} n}{1 + 2.8\, n}$, as the respective quotients are 1.23 and 1.233.

Assuming $d = 7$ ft., $l = 48$ ft., $h = 89$ ft., $E = 3 \times 10^6$, $\varDelta = \frac{l}{2\,000}$, then $S_c = 94$ lb. per sq. in.

Where the bents are $h = \frac{89}{4} = 22$ ft. 6 in., S_c is nearly sixteen times as great, or 1 504 lb. (the value of $\frac{1 + \frac{7}{2} n}{1 + 2.8\, n}$ changing only to 1.2), which is not counterbalanced by the dead load of the structure, and large cracks will be prevented only by steel reinforcement.

In this case, $E\, I$ must be replaced in Equation 10 by $E_s\, A_s d'^2$, where E_s is the modulus of elasticity of the steel, A_s is the steel reinforcement, in square inches, on the tension side, and $d' = jd$, the moment of resistance being represented by $A_s\, jd$.

The stress in the reinforcement at C

$$= \frac{2.4\, E_s\, A_s\, d'^2 \frac{\varDelta}{h^2} \frac{1 + \frac{7}{2} n}{1 + 2.8\, n}}{A_s\, d'} = 2.4\, E_s \frac{d'\, \varDelta}{h^2} \frac{1 + \frac{7}{2} n}{1 + 2.8\, n} \quad \ldots (12)$$

In this latter case, M_c is of considerable magnitude, and will cause an unequal loading of the piles, the latter being symmetrically arranged around the columns.

Mr. Mensch. Evidently the author forgot to assume a thrust at C, or he would have found that M_2, in Fig. 3 must be negative, if he considers M_1 positive.

It is absolutely unnecessary to resort to the summation methods, where the expression, $\frac{d\,s}{I}$, varies.

In most cases we can put $\frac{d\,s}{I} = \frac{d\,x}{I}\left(1 + C\frac{x}{l}\right)$, which will allow of easy integration of $\frac{M\,d\,s}{I}$ and of $\frac{M\,x\,d\,s}{I}$.

It is extremely rare, where, instead of a straight-line variation, a parabolic, or hyperbolic variation of $\frac{d\,s}{I}$ must be adopted.

The advantage of integration over summation is evident by the fact that in the first case we can obtain a formula of general application, but in the latter we obtain the solution of one particular problem only.

The use of the formulas for continuous beams for solving stresses in framed structures is accompanied with great difficulties, and is not the shortest or most lucid solution of the problem.

Any one reading the author's method of finding the wind stresses in a two-story frame, would think it requires the enormous labor of solving a six times indeterminate structure, even if both columns have the same moment of inertia. Such a structure is only twice indeterminate. On account of the symmetry of construction, the point of inflection of the girders must be the center of the span. By the same method as the writer used for the temperature stresses, the following equations may be found to solve for P_1 and P_2:

$$\frac{I_1}{I_2} P_2 = P_1 + 6\,n_1 \times P_1 - 6\,\frac{M'\,n_1}{l} \qquad \ldots\ldots\ldots\ldots(13)$$

$$0 = P_2 + 6\,(P_1 + P_2)\,n_2 - 6\,\frac{M''\,n_2}{l} \qquad \ldots\ldots\ldots\ldots(14)$$

When $n_1 = \frac{h_1}{l}\frac{I_1}{I}$, $n_2 = \frac{h_2}{l}\frac{I_2}{I'}$, and $M' = \frac{H_1\,h_1}{2}$, and $M'' = H_1\left(h_1 + \frac{h_2}{2}\right) + H_2\frac{h_2}{2}$.

Where the moments of inertia of the columns are not alike, the exact solution, as above stated, is very laborious, and mostly time and men are lacking to do such work; but, fortunately, we can find an approximate solution, by considering that P_1 and P_2 vary only slightly from the values obtained from Equations 13 and 14, and that the point of inflection of the girders moves only very slowly from the center toward the weaker column. Where the ratios of the moments of

inertia of the columns is infinite, the point of inflection would be at the connection of the girder and the weaker column; in the Twelfth Street Trafficway Viaduct it is about $\frac{3}{10}\,l$ from the weaker column. The assumption of the point of inflection is easily checked by calculating the deflection at the top of the frame. The deflection of the left column must be equal to that of the right, and if at the first trial they are not alike, a good guess will serve to make them so. Mr. Mensch.

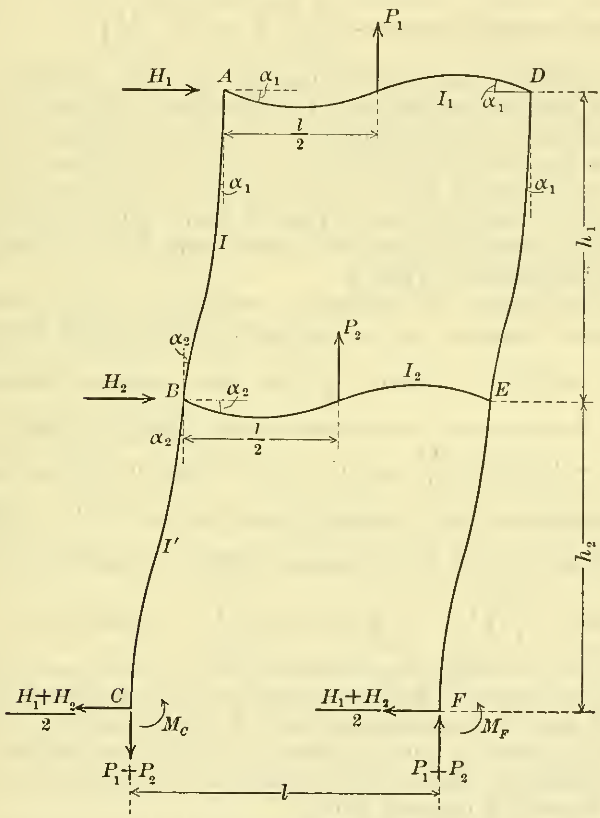

FIG. 27.

By comparing Fig. 27 with Fig. 7 it should be noted that the columns show one point of inflection at B and one midway between B and C, which were omitted by the author.

The equation for the funicular line of an arch, as shown in Fig. 8, saves considerable time over the graphical method generally used in arches with earth fill, but it is not a very fair approximation in an

Mr. Mensch. arch of open spandrel construction. In the latter case it is more correct to assume that $P_s - P_c$, the difference of weight per linear foot at abutment and crown, respectively, decreases to zero according to a straight line from the abutment to the crown, in which case the equation of the funicular polygon is

$$y = \frac{4\,h}{l^2}\,\frac{3\,x^2 + 2\,(u-1)\dfrac{x^3}{l}}{2+u},$$

and the corresponding thrust $= \dfrac{P_c\,l^2}{8\,h}\left(1 + \dfrac{(u-1)}{3}\right).$*

The summation method of finding the statically unknown value of the thrust, reaction, and moment of the abutment, requires very lengthy computations, with great likelihood of numerical and other errors. The writer can state positively that such errors must have been made when finding the influence lines, as the apex of the influence line for moments due to vertical loads at Point 6 should have been higher than at Point 8, on Plate XI; and the apex at Point 0 should have been at least 8% lower than at Point 2.

It is well known that Howe and many other writers have given very simple formulas for the statically indeterminate values of a parabolic arch with constant $\dfrac{d\,s}{I}$; but there nearly all writers have stopped. The question is often asked: can these formulas be applied to arches with varying $\dfrac{d\,s}{I}$, and of correct shape to suit the earth fill? In most cases they can be applied directly for live loads, but they fail widely for temperature and shrinkage stresses. As indicated before, $\dfrac{d\,s}{I}$ may be put $= \dfrac{dx}{I}\left(1 + C\,\dfrac{x}{l}\right)$ and, instead of a parabolic curve, the one given by the author accompanying Fig. 8 can be used. The work necessary to integrate the elastic equations is not greater than the numerical work of the summation method, and the results are formulas of general application, for any value of u, or for any relation of rise to span, making the much vaunted difficulties of arch design nearly as small as those of a common girder.

The writer cannot agree with the author that the abutment of the 130-ft. arch had to be increased over the present design in order to resist the thrust of the arch. In an arch of the shape shown in Fig. 28, the abutment, if reinforced, need not be of more than twice or three times the crown thickness of the arch in order to resist the

* The equations for both cases might be found in the writer's "Reinforced Concrete Pocketbook", where also the ordinates of the curves are calculated in fractions of h for various values of u.

moments induced by the arch action. Such an arch has the advantages of considerably reducing the temperature and shrinkage stresses over a design where the abutments are so heavy that they may be considered immovable at the top. The only difficulty is in the foundations, but also here, reinforced concrete allows of much smaller dimensions than formerly, where only massive concrete was used, and, besides, it was possible in the viaduct to consider the first adjoining spans at each side as frames, thus dividing the thrust and moments at the abutments among four piers. The great depth of the arch in relation to the rise would have caused the stresses due to drop of temperature and shortening of the arch, if heavy abutments were used, to be many times larger than the dead and live-load stresses, and it certainly was fortunate to reduce these stresses by the scheme adopted by the author, after he omitted to take advantage of the arch scheme shown in Fig. 28. The shortening of the bottom chord for live-load stresses, however, brought new stresses into play in the arch, when no live load is acting on the arch; and their values should have been considered in the design. Mr. Mensch.

The use of steel having an elastic limit of only 35 000 lb., instead of 50 000 lb. per square in., increased the cost of the structure about $40 000. The writer cannot understand the prejudice of engineers against the use of a higher grade of steel, made of billet stock, in reinforced concrete construction. All tests made by the best-known authorities show that the ultimate carrying capacity is increased by its use by more than 50% over low-carbon steel. There certainly is a very small chance that shock will affect the steel in columns or footings, or that the small steel bars in the slabs will be affected by shocks. The only place where an engineer may properly doubt the advisability of using high-carbon steel reinforcement is in the bent bars of girders, because the bars are often injured by careless bending. Where careful bending is done, with large radii, say about six to ten times the diameter, no engineer need be afraid of any unfavorable results. The writer used high-carbon bars for the reinforced floors of the Utah copper concentrator, in Garfield, Utah, and although about 8 000 tons of material have been vibrating 1 000 000 times a day, without interruption since 1907, not the slightest sign of failure has been detected.

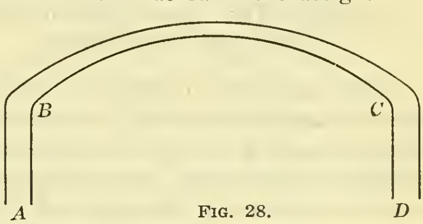

FIG. 28.

The provision in the specifications requiring bars to be firmly bound and tied together where they lap was unfortunate. The stress in bars is transmitted by shear and bond, and where bars are tied together the bond surfaces are decreased.

Mr. Mensch. The author deserves the thanks of the Profession for offering this paper, and especially for the detailed description of the methods of construction.

Mr. Holmes. HOWARD W. HOLMES,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer is very much interested in this paper, not only on account of the several uncommon features involved in the design of the structure, but because it is not often that structures of the magnitude and importance of the Twelfth Street Trafficway Viaduct are described as fully and clearly, particularly with reference to arch design and construction features. Publications of this kind are of great value to all members of the Profession interested in the design and construction of modern bridges.

In reviewing the details submitted, the writer was particularly impressed with the following features:

First.—The unequal cross-sectional area of the main longitudinal girders and arch ribs;

Second.—The curved tension reinforcement in the longitudinal girders and cross-girders.

With reference to the first mentioned, it would appear that members of unequal size would be dissimilarly influenced by temperature changes, causing thereby undue stress in the columns and floor system which would combine with the secondary stresses induced by the rigid connections of the upper and lower decks. The arch ribs, owing to their unequal size, would undoubtedly, when subjected to temperature changes, exert a like influence on the spandrel columns and supported deck, resulting in strains and cracks in the floor system and weaker members, though granting certain elastic deformation throughout the system. The stress induced as above, combined with the possible variation in temperature between the arch ribs and lower chords, should, in the writer's opinion, merit consideration. The possibility of the dissimilar temperature influence is further emphasized by the fact that the lighter members are all on the south, or more exposed side of the structure. The effects of these stresses might be modified, but not eliminated, by constructing expansion joints at the intersection of the arch axis and the line of temperature thrust, or at the spandrel columns nearest this intersection.

It is gratifying to note the thorough and systematic methods followed in the arch analysis, and that the method of influence lines is coming into more general use, as it is certainly the only method that can be relied on for structures subjected to the severe concentrations imposed by modern traffic. Under the common loadings of several

* Portland, Ore.

Mr. Holmes.

years ago, it was thought sufficient to consider the arch under two or three conditions of loading. This method, though still used to a great extent, is entirely inadequate, and, if applied, gives no assurance that the maximum stresses in the arch have been determined. Failures have been averted only by providing an excess of material; but, at the present day, when economy of design is considered a factor of major importance, a more exact method of loading must be adopted. Since the position of the live load producing maximum moment at any point will vary in arches of different proportions and form, modern practice makes it imperative, in the interests of safety and economy, that the more complete and exact method of analyzing an arch for a load of unity at each load point be used. Influence lines may then be drawn for both the upper and lower fibers, and the exact maximum values due to both vertical and horizontal loads, either uniform or concentrated, may be determined, as the influence lines will show clearly the extent and general position of the loads for maximum stress. Undoubtedly, the design of an elastic arch is one of the most delicate problems of structural engineering, but if the designer is sufficiently familiar with his work, it may be performed without great difficulty. It is only by applying rational and systematic methods, such as were used in the design of the structure under discussion, and combining the economical design evolved with faultless construction, that the field of usefulness of this esthetic form of structure will become more generally recognized.

The author has used the method of constructing influence lines for moment and thrust. By constructing these lines for stress in extreme fiber, a great deal of time may be saved by applying a modification of the stress equation for homogeneous beams,

$$S = \frac{M}{\frac{I}{C}},$$

where S = stress in extreme fiber,
M = maximum sectional bending moment,
I = moment of inertia,
and c = distance to neutral axis.

When the fiber stress is wholly compression, and considering the additional factors,

r = least radius of gyration,
N = thrust normal to the section,
A = area of transformed section,
I' = moment of inertia of transformed section,
and e = eccentricity,

Mr. Holmes. the unit stress in the concrete will be $\frac{N}{A}$, and the unit flexural will be $\frac{M\,c}{I'}$. Considering the combined section, we have

$$(M\,c\,A) + (N\,I') = S\,A\,I',$$

$$\text{or } S = \frac{N}{A} + \frac{M\,c}{I'}.$$

Equating $A\,r^2 = I'$, and $M = N\,e$, we have

$$S = \frac{M'\,c}{I'}, \text{ where } M' = N\left(e + \frac{r^2}{c}\right).$$

Computing the values of $\frac{r^2}{c}$ for the various sections, and considering this distance as the moment arm, and the fiber stress as varying as the moment, an influence line drawn for the moments thus computed will serve directly as an influence line for fiber stress. The writer has found that this direct method eliminates many chances for error, and saves considerable time. Of course, when the tensile stress in the concrete is ignored, as it should be, and the resultant stresses are tensile, it will be necessary to plat the influence lines for moment and thrust, and compute the stress in extreme fiber in the usual manner.

In connection with the discussion of stress in an arch of this type, a most important feature is the method of construction. The writer believes that cracks and initial stresses, as often developed, may be traced directly to the construction methods, and a description of the latter would complete this valuable paper.

With reference to the curved tension reinforcement in the longitudinal and cross-girders, it may not be unreasonable to ask why the main tension reinforcement was not placed in a horizontal position, and lighter and shorter lengths of steel bars used to reinforce the haunches. It appears to the writer that the method used does not accomplish the results in the most economical way. Although the distribution of metal throughout the girders is ideal, it would appear that the same results could be accomplished with a smaller quantity of steel.

As shown by the following demonstration (see Fig. 29):

Let P = total stress in bar;
p = normal pressure per linear unit;
ds = single element.

Mr. Holmes.

Then the vertical component $= p\ ds$; cos. $\theta = pr\ d\ \theta$; cos. $\theta = X$.

$$\Sigma X = \int_{c}^{B} pr, \text{cos. } \theta, d\ \theta + c = pr \text{ sin. } \theta + c.$$

$$\Sigma X = 0 \text{ when } \theta = 0, \text{ therefore } c = 0.$$

$$\Sigma X = pr \text{ sin. } \theta \ldots\ldots (1)$$

$$P = pr, \text{ or } p = \frac{P}{r},$$

substituting in Equation (1)

$$\Sigma X = \frac{P}{r} r \text{ sin. } \theta = P \text{ sin. } \theta.$$

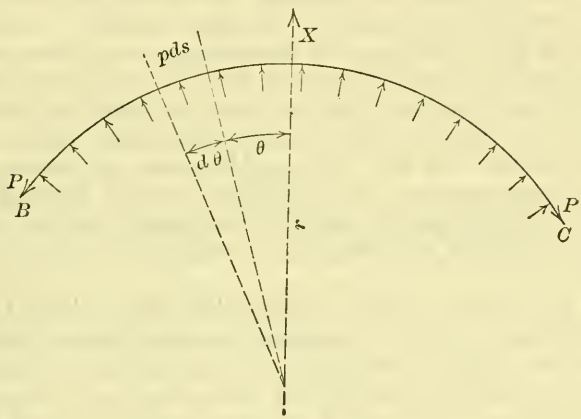

Fig. 29.

The steel with its convex curvature must resist a tensile stress induced by the bending moment caused by a vertical load equivalent to the summation of the various vertical components of force, in addition to the bending moment caused by the dead and live loads. It would seem logical to conclude, therefore, that if just sufficient steel is used to satisfy the stress induced by the live and dead loads, the working stress would be exceeded by a quantity equal to the increment of induced stress indirectly caused by the convex curvature.

On the other hand, it would not be economical to compensate for the effect of the vertical components of force by increasing the sectional area of metal at the critical section.

The distribution of the summation of the vertical forces referred to would follow the sine curve and result in considerable increase of moment at the center of the girder.

The painstaking care exercised by the designers in the close attention to details is to be commended, as there is very little to be guessed at by the prospective builder in figuring his estimates of costs.

Mr. Holmes.

Another important feature, in the writer's opinion, was the exclusion of piles in which fresh or unset concrete is placed against the soil. Considerable experience in connection with concrete piles under various subsoil conditions has demonstrated conclusively, to the writer's satisfaction, that reliable results in this most important feature of foundation work may be expected only by using piles in which the unset concrete is not allowed to come in contact with the subsoil. The many failures resulting from the use of concrete piles constructed in immediate contact with the soil should fully justify the statement that those of the latter type are not to be recommended where permanent and reliable results are desired.

The writer is pleased to note that the conventional sand cushion under the wood block pavement has been omitted. Considerable experience on the writer's part in constructing wood block pavements on bridge floors and reconstructing the floor systems of various bridges has demonstrated that the blocks are bound to heave and buckle, due to the shifting of the sand cushion, particularly when laid on a grade. The sand cushion will also give trouble where the blocks are laid with open joints, owing to the possibility of water leaching through the joints and displacing the cushion. The method used by the author has many advantages, and effectively water-proofs the bottom of the blocks.

Owing to the great variety of traffic that will no doubt be carried by this viaduct, it will be interesting to note the results obtained, under various climatic conditions, by constructing a wood block pavement on a grade of 5.5% and using the ⅜-in. lath separators between the rows. This method has been tried before, but it would seem that observation of the great variety of types of street users that may be expected to pass over this structure will result in interesting and valuable data.

In reviewing the paper, one cannot help being most favorably impressed with the fact that, in drafting the specifications, an effort was made to eliminate as far as possible the uncertain characterization of work "satisfactory to the engineer", to make perfectly clear to the prospective builder just what was to be required, and to express the views of the engineer in advance. The writer does not wish it to be inferred, however, that the engineer should have no discretion whatever in regard to carrying out the contract, but it may be unhesitatingly stated that this feature is one of the most important matters before the Engineering Profession to-day. The "satisfaction" of an engineer is a factor over which the contractor has no control, and it would be possible for a contractor to perform work absolutely above criticism, and yet not be able to compel the engineer to be "satisfied". Most contractors take pride in their work, and, as a matter of business policy and to maintain a reputation as a reliable contractor, make

conscientious efforts to perform their work to the satisfaction of the engineer. Reliable contractors realize that competent supervision on the part of the engineer is absolutely essential. The making of a contract to the effect that work shall be performed to the "satisfaction of the engineer" and the letting of contracts without advance engineering information have been responsible for considerable pooling and combination in the contract business. The very idea of contracting to perform certain work is that such work can be planned and estimated in advance, as a result of experience with similar work, under like conditions, so that the contractors may have full knowledge of what is to be required and be able to fix with reasonable accuracy the cost of performing it. Contract work should be confined to that which involves quantities and conditions which may with reasonable certainty be determined in advance by experienced engineers. Work which cannot be determined in this way should be let on a basis of reasonable cost, plus a just percentage for profit. It should be clearly understood, however, in this latter case, just what items should be included in the cost. Mr. Holmes.

In defense of the Profession, however, it should be considered that many of the weaknesses of specifications governing, or supposed to govern, public work may not be the fault of the engineer, as it is undoubtedly true that many of the absurd and unjust requirements, which are injurious both to contractors and to the public, are provided by law and by city charters. That many of the clauses found in public work specifications are not necessary is shown by the faithful and satisfactory completion, by contractors all over the country, of contracts with business firms where no such clauses are considered necessary, and where there exists recognition of the relation between the responsibility of the contractor and the requirements for the bond and retained percentage.

One of the main reasons that public work is often more expensive and less satisfactory than work contracted for by private firms is the fact that most of our laws compel the award to the lowest bidder. This, combined with the many other uncertain requirements, has in many cases proved most injurious to the public welfare.

It is quite generally supposed that profits from public work contracts are so great that anything that can be imposed on the contractor is so much clear gain. Needless risk should not be imposed on a contractor. Any one possessing an elemental knowledge of business principles should know that no competent and financially responsible contractor will take a gambling risk without charging enough on his bid to save himself from loss. Of course, there are contractors who will bid low in order to obtain the work, but, in the majority of cases, they are either financially irresponsible, or shrewd enough to make up their loss by slighting the work. Work let to the contractor

Mr. Holmes. of this latter class is bound to cost in the long run a good deal more than had it been let to a competent and reliable man who understands his business.

The writer has had considerable experience in contract work, both public and private, and has learned to appreciate the integrity and desire of contractors to perform work creditable to their reputation. The Profession should stand as a unit in making an effort to overrule the many unfair clauses of almost universal use in specifications, as these clauses have their part in limiting competition, in lowering the standard of honesty among contractors, and in lessening the dignity and standing of the Profession. By cultivating the confidence of reputable contractors and providing for competitive bidding on public contracts, immense sums may be saved to the taxpayers. A continuation and more wide-spread adoption of the policy followed by the engineers in connection with the contract for the structure under discussion will accomplish the desired results. The author deserves the thanks of every member of the Profession for his effort.

Mr. Upson. M. M. Upson,* Assoc. M. Am. Soc. C. E. (by letter).—As is usual in a descriptive paper of this character, limited space has prevented Mr. Howard from taking up in detail the reasons for selecting the component parts of the structure. In particular, a short account of the selection of Raymond piles by the contractor, and their acceptance by the engineers, will show the care and forethought which characterized this entire work, and can now be seen to have saved both time and money.

Up to 1912, with one exception, the only concrete piles used in Kansas City were pre-cast. Mr. Howard had used pre-cast piles on a somewhat similar viaduct within half a mile of the present structure. It was natural, therefore, that the specifications should allow pre-cast as well as cast-in-place piles, the latter being limited to those in which "fresh or unset concrete is not placed against the soil." Pre-cast piles 25 ft. long were estimated as being necessary, it being provided that a variation in length could be ordered if required when the work began, and it was specified that such piles were to be jetted into place.

The soil of the site is a typical Missouri River fill overlying a very fine and comparatively clean sand, admirably suited to jetting, and into which the contractor would have experienced no difficulty in placing the 25-ft. pre-cast pile. On the east end of the viaduct, where it approaches the rock cliff, erosion subsequent to the initial placing of the soil had evidently taken place, as the sand and rock were covered with a very soft muddy material which necessitated piles at least 30 ft. long.

* New York City.

It is apparent that had pre-cast piles been used, about 10% of their total number would have been increased fully to 30 ft. in length on this account. The remainder, by the use of a jet, would have penetrated 25 ft., so that the 2 610 piles would have totaled 66 555 lin. ft. Mr. Upson.

After a careful study of the conditions, Raymond piles were selected. They were driven without jetting to a sufficient penetration to support safely the maximum load of 35 tons each. By driving to an agreed penetration to the last inch, a uniform bearing value was obtained, the lengths varying to meet the different soil conditions. They ranged from 12 to 30 ft., and in some individual piers there was a length variation of as much as 5 or 6 ft. By the use of the Raymond system, it was unnecessary to predetermine the lengths of the piles, and the city paid for only the actual number of feet driven. In this instance, the 2 610 piles totaled 46 625 lin. ft., which showed a saving of 19 930 ft. over the prescribed 66 550 lin. ft. of pre-cast piles. At the contract price, $1.20 per ft., this saved the city about $24 000, or 43% of the total cost of the piling.

It may be well to point out one or two reasons why, in the writer's opinion, a less length of pile may safely carry the same load. In this instance, as in most pile installations, the piling element does not penetrate to rock or hardpan, but carries its load by surface friction. The friction per square foot of surface in a given soil is dependent on several factors:

1.—The angle between the soil and the surface. It is obvious that the friction on a vertical surface can be no more than the shear of the adjacent soil, though the friction on a perfectly horizontal surface will be equivalent to the compressive strength of the soil. The vertical surface friction varies between these two limits. On that account a tapered friction pile has a great advantage over the straight cylindrical pile. The writer has frequently tested tapered piles to a surface friction of more than 2 000 lb. per sq. ft. without rupture.

2.—The degree of compression of the soil which is attained by the driving. This depends on the amount of abuse to which the driven pile or form may be subjected. The necessary cushion to protect the concrete of a pre-cast pile limits its effectiveness in this function.

3.—The maintaining of the compression after it has been attained through the driving. This is effectively accomplished in the pre-cast pile, as it is also in the Raymond type with its spirally reinforced and corrugated shell, which is left in the ground.

In addition to the saving of cost by reason of the shorter lengths required, the use of Raymond piles avoided a difficulty which would undoubtedly have proved both costly and troublesome to the contractor. This is the operation of jetting on the site.

Mr. Upson. The work was in a congested district, with unusually heavy street and railroad traffic. Had pre-cast piles been used, the disposal of the water from the jets would have been a great problem. A filter of some kind would have been required to separate the sand from the water, as the sewers in that part of Kansas City have a very small fall, which would prevent them from carrying water containing any large quantity of sand.

The jetting of piles in a city always entails many difficulties, especially when fine and light soil, easily carried in suspension, is encountered. It is, of course, quite impossible to carry the water on the surface, because of streets and crossings which occur at close intervals. It will be seen, therefore, that, by avoiding this difficulty, the contractor saved himself much annoyance and expense.

Time is another factor of economy affected by the use of Raymond concrete piles on this work. Had pre-cast piles been used, most of them would have been cast during the winter. A minimum time of 30 days was specified, and under ordinary winter conditions this time would have been doubled before the piles had cured sufficiently to be safely handled. As a matter of fact, the weather during this work was unusually mild, so that the temperature delay would not have been serious. However, the necessary 30 days after the receipt of the reinforcing rods, which usually requires several weeks for delivery on the ground, necessitates an interval of from 6 to 8 weeks between the signing of the contract and the placing of the footings. Furthermore, a slight variation in the underlying soil conditions may involve the contractor in a further delay of from 30 to 60 days, in order to provide longer piles to meet the varied requirements.

The actual money saving by cutting down the time is frequently lost sight of by the contractor as well as by the owner. After a construction project is determined on, and the preparations are made, the money loss to the owner usually can be computed accurately, and often amounts to several hundred dollars a day, and all wise contractors readily realize the fact that their profit, under ordinary conditions, is inversely proportional to the time required to complete the work. Several instances have come to the writer's attention where the loss to the owner and a proportional amount to the contractor, on account of the delay incident to choice of improper foundation methods, has involved a sum greater than the total cost of the foundation itself.

To summarize: Raymond piles were selected, first, because of the saving in length, which, in this instance, amounted to 43%, or $24 000, on the total expenditure of $56 000; second, the saving of time, which may be estimated at not less than 30 days, as the footings and superstructure followed very closely on the driving of the piles; third, the avoidance of annoying and expensive pumping equipment and methods for taking care of waste water incident to the jetting.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

PEARL HARBOR DRY DOCK
Discussion.*

By Messrs. Charles Evan Fowler, Harrison S. Taft, W. F. Frear, and P. L. Reed.

Charles Evan Fowler,† M. Am. Soc. C. E. (by letter).—The Profession is greatly indebted to Mr. Stanford for his comprehensive paper on the Pearl Harbor Dry Dock. Mr. Fowler.

The construction of graving docks in the United States still remains very much of a novelty, although abroad it is a very common occurrence. The very fact of their novelty, although making the paper of greater value, is undoubtedly one of the causes of trouble in their construction, as neither the Government nor the contractors are able to obtain engineers or workmen who are experienced in their design or construction.

The sites for naval dry docks are seldom chosen by the designers, except within very narrow limits, but are selected for strategic, political, or other evident reasons, entirely aside from their suitability for a structure of such magnitude. Then, again, the time allowed for designing is usually all too short for a comprehensive study of the conditions which will be encountered and must be taken into account in both the design and construction.

Imagine, too, the slight chance which bidders have to study carefully and digest the problem in the short time provided by the law for preparing and submitting bids. The bidder, of course, is in no way obligated to verify the borings, which, if made by the wash-boring process, are almost always unreliable and likely to suggest conditions which properly made core borings would show to be non-existent.

* This discussion (of the paper by H. R. Stanford, M. Am. Soc. C. E., published in May, 1915, *Proceedings*, and presented at the meeting of September 1st, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Seattle, Wash.

Mr. Fowler.

The construction of graving docks in England is very common in connection with projects for immense wet docks, and in some cases, they are constructed parallel to the lock entrances to the wet dock, as at Tilbury, so that in case of an accident to the lock, they can be used as entrances to the wet dock. Comprising as they do a portion of such a great system of construction, it is natural that more time should be afforded in which to study the conditions and design, and consequently less trouble would be encountered during construction.

After reading Mr. Stanford's paper, the writer can see no reason for changing the conclusions reached some years ago, that the proper method of constructing a dry dock where conditions are unfavorable, as was the case at Mare Island and at Pearl Harbor, would be to do it under water, which was done successfully at Kobé, Japan, and has been fully described* by the writer.

That there may be serious objections to such a method is granted, but they are not such as cannot be readily overcome by the careful designer and constructor. The paramount objection is that of obtaining good results in depositing concrete in sea water, and the writer's experience leads him to believe that, if made in this way, with proper equipment and care, the concrete will be just as good as, or better than, if deposited in the dry, and at no increase in cost.

The method used should be that known as the "Detroit Tunnel Method", and the tremie should be kept buried in the concrete at all times and moved around slowly, so that the concrete does not have to run to any great distance and does not come to any extent into contact with the water. This will obviate the washing of the cement, laitance, and consequent poor concrete.

Strange to record, perhaps, the writer's experience has shown that a smaller pipe for the tremie than any of those used at Pearl Harbor has given better results. That used at Tacoma for depositing concrete in depths between 50 to 60 ft. was of 10-in. lap-welded, screw-jointed pipe. This was successful because the interior was smooth, the joints allowed no water to leak in, and the tremie was light enough to allow it to be moved easily and properly. This work has been described by the writer,† who has demonstrated that riveted pipe is very objectionable.

Where riveted pipe has been used, it has always been larger and has had to be kept filled above water level, which is unnecessary with the smooth, screw-jointed pipe. For shallow depths, a square wooden pipe, with its smooth interior, will be found to be very good.

Although it is undoubtedly best to use a 1 : 2 : 4 concrete for under-water work, one of 1 : 3 : 5 will be found to give good results with a proper tremie and with proper care.

* "Sub-Aqueous Foundations," Chapter XXX.

† *Loc. cit.*, Chapter X.

A few years ago, and simultaneously with its use at Kobé, the writer proposed the under-water method for a large dry dock in a mud foundation, with piling cut off under water, believing it to be the surest way of proventing trouble due to upward pressure. Practically all the weight is available when the dock is pumped out, the anchorage of the piling is effective, and a steel mat of reinforcing bars can be placed in under-water concrete, if necessary, to resist the arch action in a wide dock. Mr. Fowler.

Experience has shown* that where piles are driven with a cast-steel follower cap to which their heads are fitted, they are left in better shape than when cut off under water.

The box type of coffer-dam was proposed for the case mentioned, such as that used at Mare Island and that first used at Pearl Harbor, but the very great care necessary to build a fairly tight dam and the danger of just such a failure as was realized at Pearl Harbor, would seem to make it a grave question whether its use is ever advisable.

Conditions might be such that neither the under-water nor the box coffer-dam method would seem advisable, and it might then be asked, are there no other plans of construction possible than the phenomenal scheme proposed by Mr. Noble or the monumental one that has been adopted? Although evidently there are many answers possible to such a question, the logical one would seem to be that a movable pneumatic caisson or diving bell might be used to construct the dock in sections, joining them together with tremie concrete, as proposed in the Gayler block design, or by the use of a movable joint coffer-dam, as proposed for the adopted method, and which presumably is similar to that used for the Antwerp Quay Wall.† Although the Noble plan and the adopted plan are undoubtedly sure methods, and for this reason may prove the cheapest, it does not seem to be presumptuous to call attention to the foregoing ideas which may prove worthy of adoption in some future case.

The vital points are that more time should be allowed for the study and design of such large structures, and that more time should be given the bidders in which to study and estimate on the work and submit proposals.

Harrison S. Taft,‡ Esq. (by letter).—In view of the article entitled "Floating Concrete Caissons" by the writer and Mr. O. D. Jones,§ Mr. Stanford's paper has been read with the deepest interest. Notwithstanding the varied uses that have been made of floating concrete caissons, the work now being done at Pearl Harbor under the direction of Mr. Stanford's department stands out as one of the most extensive Mr. Taft.

* *Loc. cit.*, p. 202.

† *Loc. cit.*, Fig. 427.

‡ Seattle, Wash.

§ *Professional Memoirs*, March–April, 1915, p. 145.

Mr. Taft. and most scientifically worked out applications of the art, up to the present date; and its successful accomplishment will be another triumph for American engineering. Although a lengthy discussion of the floating concrete caisson problem by the writer would be superfluous, there are several points in this worthy paper on which, it is thought, a few words can well be said.

Mr. Stanford mentions the serious defects found in the concrete first placed under water by tremie operations, and seems to lay special emphasis on the desirability of excluding salt water from within a mass of freshly deposited concrete. As is well known by those most experienced in placing concrete under such conditions, not only does the infiltration of sea water into it cause the formation of the white chalky substance which prevents the proper formation of the plastic concrete into a dense, water-tight, and homogeneous mass, but inexperience in the operation of the tremie leads to results that are practically worthless—that is, in a great many cases. It is of interest to note that, in the construction of a graving dock by Japanese engineers, special attention was given to the problem of preventing the intermingling of sea water with the green and freshly placed concrete in depositing, under water by the tremie process, a thick layer of concrete as a "seal" for the base of the floor of the dock. The method they used was the reduction of the saline contents of the sea water to a minimum by the introduction into the enclosed coffer-dam, of fresh water from the city water supply, an operation that is said to have met with marked success, and resulted in a first-class concrete sealing job. The coffer-dam was afterward unwatered and the remainder of the dock was completed in the dry with equally marked success. Whether such a mode of procedure would have been possible, or would have eliminated some of the troubles at Pearl Harbor, the writer is not in a position to state.

Mr. Stanford's statement that briquette tests showed that concrete mortar must not be of a less than 1:2 mixture, for a successful use under sea-water conditions, coincides with the results of other experiments by prominent engineers.

The writer notes that Mr. Stanford does not state the chemical composition of the cement being used at Pearl Harbor; it would be of interest if he would furnish some data as to whether he is taking any steps to obtain a special manufactured cement for the work, or is using the daily commercial output of the mill.

The author mentions the rapidity with which the concrete was mixed, namely, batches, of some 1.85 cu. yd. each, at the rate of 40 to 50 per hour, equivalent to a batch every 72 to 90 sec. (approximately $1\frac{1}{4}$ to $1\frac{1}{2}$ min.). Though it seems to be the aim of some engineers, and especially of contractors, to turn out the greatest possible quantity of concrete per hour, without regard to the time of actual mixing, a

great many are beginning to feel, and note in their specifications, that some limit should be put on the length of time a batch is kept turning in the mixer, especially when the concrete is to be exposed to contact with sea water, and still more especially when placed by a tremie. Though a 5-min. mixture may be excessive, it is without doubt far better to mix the concrete as long as 5 min., or even longer, than to shorten the time of the mixing to such an extent as to have the supposedly mixed concrete discharging before the last of the unmixed ingredients have been put into the mixer, a mode of procedure that most surely will cause future troubles, and in a very short time at that, if the concrete is used in structures in contact with sea water. A period of fully 3 min., as called for by some engineers, or a definite number of turns of the machine, such as 25, as used on one Government job of magnitude (all reckoned from the time that the last of the necessary ingredients have entered the mixer), will no doubt give far better concrete than the speed quoted by the author. It is possible that the rate of mixing, as well as the lean concrete, and the inexperience in using the tremie, account for the poor and worthless results Mr. Stanford says were obtained with the concrete placed in Crib II. Mr. Taft.

The writer has at times been asked as to the holding-down power of a pile having its head encased in concrete, and as to whether the head would be pulled out of the concrete or the pile pulled out of the ground by the lifting of the concrete structure. One case, at least, which has been cited, was the washing out of a bridge by flood waters, whereby some of the piles were broken off at the under side of the concrete base and others were pulled completely out of the earth and remained embedded in the concrete as when the structure stood erect in its proper position. As well known, the figure usually adopted to determine the carrying power of a pile in compact sands is about 660 lb. per sq. ft. of pile surface, equivalent to about 1 ton per lin. ft. for a 12-in. pile. On the basis of 47 tons upward pull, as noted by Mr. Stanford, and a 5-ft. embedded head, for a 12-in. butt, the calculated resistance is about 3 tons per sq. ft. of contact surface. The resistance, for a pull to destruction of the bond, is apparently unknown.

In giving the results of the experiments with a tremie, Mr. Stanford has added valuable material to a subject on which engineers in actual charge of work have had but few accurate data. In several articles in engineering literature during the past few years, facts of considerable worth have been given to the contracting engineer, as to the best size of tube, etc., and as to operation; but all such data, even that of Mr. Stanford, do not seem to have furnished the engineer with a suitable description of the method of first loading the tremie, in order to prevent absolutely the intermingling of the concrete and the water in the tube. In other words, there is no improvement over the old-fashioned methods of driving the water out of the tube during

Mr. Taft. the loading period, and at the same time keeping the initial loading of the tube and its going into operation under absolute control, irrespective of the depth of the water or the size of the tube.

In depositing the subaqueous concrete around the tubes of the Detroit River Tunnel, "a wadding of cement sacks was at first placed in each tremie on top of the water to prevent the concrete from dropping through while filling the pipe".* Batches of concrete were then poured in on top of the wadding, the slight raising of the tremie permitting part of the water to run out and the stopper to run part way down the tube. More concrete was then poured in, etc., until the stopper finally passed out of the bottom of the tube and the outflowing concrete formed a seal around it. After some tests it was found that dry concrete could be used to take the place of the sack wadding, and such a mode of procedure was adopted after the work was well under way.

In place of a sack wadding as a stopper, some engineers have used paper bags filled with dry concrete; others have used a wadding made by dumping in a pail of sawdust on top of the water before pouring the concrete into the hopper of the tremie. Mr. G. W. Rear has devised a mode of using a stopper consisting of some 7 or 8 in. of planer shavings covered with 1 in. of cement, the buoyancy of the shavings and the resistance of the column of water below preventing the stopper from falling too rapidly down the tube, the concrete itself acting as the seal. As stated by Mr. Stanford, at Pearl Harbor, a bag filled with straw was inserted and descended in the tube when the concrete was placed above it.

Although the foregoing and other methods of loading a tremie usually call for the assembly of a large quantity of concrete for the initial pouring, they also leave the operation practically uncontrolled by the men in charge, and the discharge of some tons of weight into the tremie at one operation is apt to set up considerable vertical movement, especially when the tremie is hung from the end of a boom on a scow, thus interfering seriously with successful loading.

The introduction of such matter as sawdust, shavings, straw, paper bags, etc., may be of absolutely no importance in the placing of subaqueous mass concrete, yet in cases of reinforced concrete columns, from 3 to 4 ft. in diameter, as used at San Francisco and in the Puget Sound Navy Yard, the introduction of such foreign matter may affect seriously the permanency of the structure. The elimination of all foreign matter from concrete placed in sea water should be absolutely insisted on, if the best results are to be guaranteed. It was in connection with just such a guaranty that the writer was led to devise an all-concrete stopper, mechanically controlled in its descent through the tube, even though the lower end of the latter was far

* *Engineering News*, March 17th, 1910, p. 320.

above the mud line and freely swinging in the water. A brief description of this stopper, it is thought, will be of interest. Mr. Taft.

As shown in Fig. 20, the stopper is of concrete, having a circular disk for its base and a frustum of a cone for its head, the two parts being made monolithically. It is built in two parts, with a hole at the center, so as to permit them to be placed around a bolt and rest on a washer at the pointed end. The diameter of the stopper should be such as to suit the tube, with sufficient clearance to permit of a free descent. The bolt is suspended by a wire or rope running to some kind of controlling device, such as a winch, to provide for

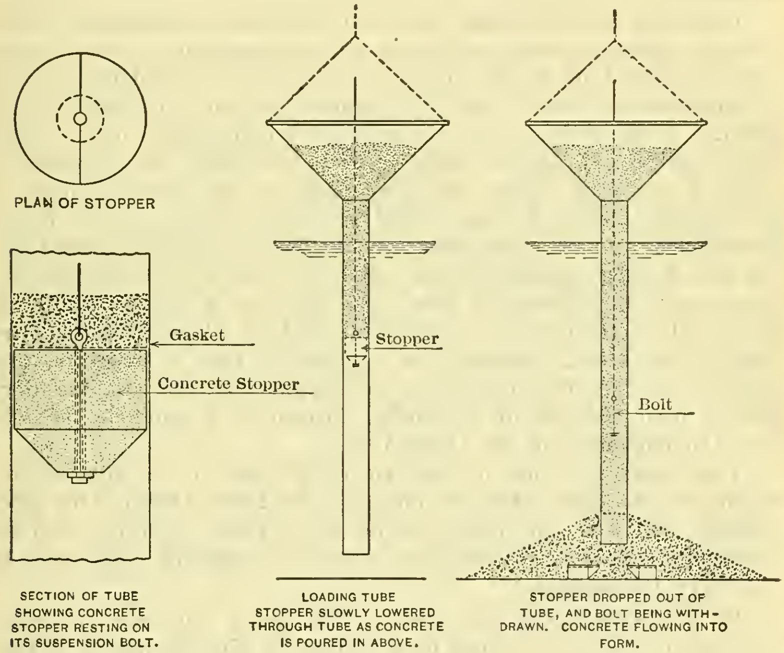

FIG. 20.

lowering the stopper through the tube. If desired, a plastic washer of soft leather or rubber may be placed at the top of the stopper, as shown. The two parts of the stopper are first placed around the bolt in the upper part of the tube and lowered slightly. Concrete is then poured in above, and the stopper is slowly lowered as more concrete is poured in above it. When the stopper has reached the lower part of the tube and is ready to pass out, the tremie having been raised sufficiently above the ground, the unbalanced couple formed by the supporting bolt and the weight of the concrete above the stopper causes the two halves to fall apart and permit the outflow of the concrete to

Mr. Taft. form a seal about the bottom of the tube, the hopper having been fully loaded in order to prevent any interruption in the column of concrete in the tube after the opening of the stopper. The bolt is then withdrawn and kept for the next loading. As the two parts of the stopper remaining in the mass are of concrete, there is no foreign matter in the deposited concrete, as is the case with some of the older modes of operation. Though the writer has not yet had an opportunity for an extensive trial of this device, it appears evident that it provides an absolute means of preventing any intermingling of the water and the concrete, and at the same time there is positive control of the whole operation.

Inasmuch as the prime factor in securing a successful use of concrete under sea-water conditions is a pre-moulded, air-cured structure, as pointed out by the writer,* there can be no doubt that the use of structures of such a type by engineers is bound to become more general, as the years go by, the successful culmination of the Pearl Harbor project, no doubt, doing much to encourage the engineers of America in a still further and more extensive use of such a mode of construction and in a more liberal utilization of concrete in harbor development and sea structures. Though it is a matter of record that the first floating concrete caisson was built without any reinforcement whatsoever, all subsequent ones have been on a typical reïnforced basis. The extensive use of structural steel, as adopted for the Pearl Harbor Dry Dock, apparently is the first in such a structure, thus marking the fourth step in the development of the art, the third having been the use of a wooden, instead of a concrete, base by Mr. Tromanhauser on the Great Lakes.

The congratulations of the entire Profession are due to the engineers who have solved so uniquely the Pearl Harbor Dry Dock problem, and will still further be due to all those connected with the successful completion of the work when it is officially taken over by the Navy Department.

Mr. Frear. W. F. Frear,† Esq. (by letter).—The writer has read this paper with much interest as he had long connection with the matter as legal advisor of the contractor and, of course, incidentally had more or less to do in connection with the engineering and financial aspects of the matter. The paper seems to present the subject lucidly and also fairly, although a paper written from the contractor's standpoint would naturally differ from this in some respects. The writer does not think he can say anything of sufficient value to warrant comment on the paper, further than, perhaps, to say that the statement of the opinion of the Attorney General with reference to the responsibility of the contractor may be misleading, although, of course, unintention-

* In "Floating Concrete Caissons."

† Honolulu, Hawaii.

ally so. A reading of the entire opinion and not merely the quoted extract from it would show that the Attorney General held that the contractor was bound to complete the structure according to the plans and specifications, but not bound to produce a stable and satisfactory dry dock; in other words, that the contractor's obligations would have been fulfilled if the structure had been brought to completion according to the plans and specifications even though it should collapse afterward. Mr. Frear.

It was a matter of gratification, both to the Government and the contractor, to be able to arrange for the completion of the work on the new plan under a supplemental agreement, and it is to be hoped that in due time a dock satisfactory to all parties will result.

P. L. Reed,* M. Am. Soc. C. E. (by letter).—Supplementing the detailed account of the Pearl Harbor Dry Dock given by the author, as regards location, geology, history, finances, design, and construction so far as it has progressed, a few general features may be emphasized. Mr. Reed.

There have been two primary problems in the design of this structure: First, to design a dry dock which would safely dock ships of specified maximum dimensions, and would withstand indefinitely the stresses brought on it in this service; second, to design a dry dock which could be safely constructed on the Pearl Harbor site.

In this case, the first problem was much the simpler. Given the interior clearances, it is required to design a concrete box which will weigh when empty approximately as much as the water it displaces at high tide, in order that it will not float, and which, in this condition, will safely withstand the inward pressure of earth and water on its sides and bottom. A transverse section or slice of the empty dock is, in fact, an inverted arch, and the lines of pressure follow a similar course. When a ship is placed on the blocks, its weight offsets a portion of the upward pressure on the bottom, and the principal stresses on the section are reduced. When the dock is full of water, the stresses in the concrete act on different lines, but are less exacting. It so happens that with the quantity of concrete required to balance the displaced water and thus offset the buoyancy, it is not difficult to obtain a cross-section of concrete without steel reinforcement, which will safely carry the stresses, assuming uniform pressure on the bottom and reasonable earth pressures on the sides.

It will be noted that, when the dock is empty, it is practically floating and brings no appreciable load on the bottom. When full of water, the load on the bottom, in the case of a dock of these proportions, becomes about 1 ton per sq. ft. uniformly distributed, and the plain concrete section can effect this distribution without inducing undue stresses, still assuming reasonable earth pressures on the side-

* Washington, D. C.

Mr. Reed. walls. It will be seen from this that the maximum load on the bottom is very moderate, in fact, as indicated in the paper, it is less than the overburden of earth removed in excavating the site from an original level surface, so that, even with a poor bearing material, there can be little danger of serious settlement. So much for the first problem.

The second and much harder problem was to design such a concrete box that it could be positively and safely constructed in a freely water-bearing soil where a coffer-dam was found unavailing, as the water came up through the bottom. As the empty concrete box weighs as much as the water displaced when it is in place on the bottom, it could not be constructed in whole or in complete sections and floated to place, as has been done with some concrete caissons and sea walls, and it is very undesirable to deposit concrete under sea water where it can be avoided. What has been done in the adopted design, as explained in detail in the paper, is to provide a portable floating steel coffer-dam by which transverse sections or slices of the dry dock can be partly constructed on a floating dock, floated off, sunk in place, and held down by a tank of water while the sections are being completed in the dry. This method of construction brings in temporary stresses which are greater than those which the finished dock must sustain, and a large quantity of steel framing and reinforcement is required, which remains in the finished dock and increases its strength correspondingly, but which would not be necessary, except for the unusual method of construction required.

The steel coffer-dam combined with and surrounding the steel box which serves alternately as a water tank to hold down the uncompleted, and therefore buoyant, sections of the dock, and as a float to lift the coffer-dam as a whole from a completed section and place it on a new dock section partly constructed in the floating dock, is the ingenious invention of the author to meet the unusual conditions described, and constitutes an effective and novel combination of construction appliances.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

RIVERS AND RAILROADS IN THE UNITED STATES

Discussion.*

By W. W. Harts, M. Am. Soc. C. E.†

W. W. Harts,‡ M. Am. Soc. C. E. (by letter).—It seems to be a rather familiar criticism, not altogether unfounded, it must be admitted, which has been advanced by students of the development of our rivers as common carriers, that we do not accomplish as much with our money here as some foreign countries where the systems of selection of waterways for improvement and the amounts expended are the subjects of what is supposed to be much more careful scrutiny than here. It was the writer's purpose to point out how some of the economic features of our systems might be eliminated and the profitable features developed further, all with a view to obviating in the future the justice of some of these criticisms. Mr. Harts.

It should be kept in mind that in most foreign countries the choice of what rivers to propose for traffic purposes is by Nature already made, in most cases there being but few from which to select. Furthermore, the conditions surrounding their use there are so different from what they are here that the inestimable service that many of our rivers have already rendered in the development of a new land is likely to be overlooked when their present idle condition is in mind. Most foreign countries charge a tonnage tax on all-water freight. Many of them allot a large part of the first cost of construction to localities benefited. Whether or not these requirements are wise, in the long run, the river navigation of this country is free of both these burdens. Then the density of population is much greater there than here; the rivers were used for commerce, and the people grew to be accustomed to them long before railways were the efficient carriers they have

* Discussion of the paper by W. W. Harts, M. Am. Soc. C. E., continued from August, 1915, *Proceedings*.

† Author's closure.

‡ Washington, D. C.

Mr. Harts. become of recent years; and furthermore, the railway systems there are in the main so subject to Government control that they are often compelled by order to fix their rates so that the rivers will get some undue preference. These considerations put a somewhat different aspect on the economics of this whole subject.

Few persons who are well informed on river improvement will begrudge the greater part of the expenditures made for interior rivers in the past when they consider the great effect on the expansion and growth of this immense country, but what should seem wise to them now would be to scrutinize anew our waterways and select for our present expenditures only those the future usefulness of which is assured. For the future, it seems plain that we should restrict expenditures on those like the Kentucky, the Missouri, and the Mississippi, on which the conditions of channels for navigation are already far in advance of the necessities of present commerce. In this way we may even yet remedy some of the present defects and increase our average of efficiency.

Mr. Lavis seems to feel that our system of channel building is haphazard. Perhaps this idea may arise from some misconception of the way many of our streams were developed. The whole growth of our land might, in the same sense, be considered to have been haphazard, but under the visible surface the little understood but inevitable law of economics has been working incessantly and surely. The wish of the writer now is to apply the same law of economics to the future of our river systems and recognize more clearly the limits of the work required. The Board of Rivers and Harbors, through whose hands every new project must pass for examination and recommendation, is already restricting the expenditures for certain streams, but the people as a whole must see the reason for this and must support it if this work is to be done efficiently, economically, and wisely. It is not very difficult to determine the worth of any river for navigation, although predicting its future is a much harder task; but, like any other engineering project, intelligent analysis will point the safe way.

Major Burgess is correct in his discussion. Railroads should not be permitted to discriminate against rivers by favoring one locality at the expense of others, for this, as he has so well pointed out, is contrary to good public policy. Happily this tendency is believed to be proving less important, and the control of rates by the Interstate Commerce Commission may, it is thought, be trusted ultimately to regulate even this unfairness. Already the milling in transit privilege is ended, and other restrictions will probably soon follow, if unfairness exists. It is too much to expect all these new changes to be made at once, but the steps already taken may be assumed to be a good index to the future, and the future is what we must keep in view.

There is everywhere a large class of critics, and they impatiently call on other people for the immediate advantages of a Utopian prospect where everything is as it should be. Whether these dreams would turn out to be as successful as the anticipation, only a trial could prove. It seems safe to say that it is usually the part of wisdom to point out the improvements that are practicable in the existing order of things, thus building with assurance of benefit on what already is well under way, rather than to pull down everything for a fresh start. Thus we may often avoid the dangers of the radical and hazardous uprootings of the eager but short-sighted reformer. Mr. Harts.

Mr. Bernhard thinks we have too many sizes of locks and channels. He thinks that the standard railroad track and car point out a lesson for our river engineers. This is probably an unconsidered reflection. Economy would certainly indicate the unwisdom of the construction of as large locks and as deep and wide channels on smaller tributaries of the Ohio, for example, as those in the main stream. Standardizing is desirable, up to a certain point, and the present varying sizes of locks and dimensions of channels are perhaps greater than desirable, but to give the impression that there should be a standard lock or standard channel and a uniform boat suitable for all purposes is manifestly following the railroad analogy too far. Mr. Bernhard also charges lack of terminals with being responsible for the diminishing inland navigation. Adequate terminals are necessary on all transportation lines, but they do not make traffic; they only accommodate it. If commerce were active enough on our interior rivers, terminal facilities would keep pace with its requirements. We may be sure of this, from the inevitable economic law. The lack of suitable terminal facilities is only an indication of the lack of the necessity for them. On the Great Lakes, where a large commerce is found, terminal facilities are more advanced than in many of our sea ports.

With regard to Mr. Bernhard's conclusions, many of them would doubtless be desirable, but others are open to objection. The creation of a Federal Waterways Bureau or department has been suggested many times before. Can any one believe that a new department with a Cabinet officer at its head would lessen those burdens which river transportation is now struggling under? Whenever there is again a sufficient profit in river traffic, we shall see new and suitable terminals, comprehensive plans for river development, improved steamboats and barges, and a rejuvenation of an old industry; but, until that day comes, we may discuss the case often and long, and ply the patient with all sorts of supposed remedies, but the trouble will be found to be unimproved until the economic conditions on our streams grow better.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JOHN CHARLES WILLIAM GRETH, M. Am. Soc. C. E.*

DIED AUGUST 7TH, 1915.

John Charles William Greth, the son of William and Caroline Greth, was born in Buffalo, N. Y., on May 15th, 1874. He attended the public schools of that city and was graduated from the Central High School in 1893. In the same year he entered Cornell University, from which he was graduated in 1897 with the degree of Mechanical Engineer.

After his graduation, Mr. Greth began his engineering work by installing, and operating for a few months, a power-plant at one of the summer resorts on Lake Erie. During the succeeding years, from 1898 until 1902, he successively installed pumping machinery, designed special machinery, operated a power-plant, and installed and operated refrigerating and ice-making plants.

In February, 1902, he entered the service of William B. Scaife and Sons Company, of Pittsburgh, Pa., as Manager of the department devoted to the softening and purification of water. From that time until his death his whole energy was devoted to the development of apparatus and methods for the softening and purification of water for all purposes. He was granted sixteen patents on improvements in water-purifying apparatus, several of which marked radical steps forward in the development of mechanical methods in water purification. In this field Mr. Greth occupied a dominant position, and his forceful presentation of the subject contributed very materially to the advancement of the science and to the broadening of its application for various industrial uses.

He contributed articles on that subject to the technical publications, and read a number of papers before various engineering societies. He was recognized as an expert in this art by engineers and chemists, and his sterling integrity gained for him a wide circle of friends and admirers. Mr. Greth was noted for his frank and friendly disposition, and numbered a host of friends among his acquaintances. By his sincerity and forceful personality, coupled with marked ability in his field of endeavor, he commanded the respect of all with whom he came in contact.

He had not been in robust health for several years, and the shock and grief over the loss of a two-year old daughter on July 14th brought

* Memoir prepared by M. F. Newman, Esq., Mgr., Water Purifying Dept., Wm. B. Scaife and Sons Co., Pittsburgh, Pa.

on a nervous break-down, from which he did not have the strength to recuperate, and, after an illness of two weeks, he died on August 7th, 1915, at his summer home at Gibsonia, Pa. His remains were buried in Allegheny Cemetery, Pittsburgh, Pa.

In 1902, Mr. Greth was married to Miss Laura Heussy, of Buffalo, N. Y., who, with two sons, aged 11 and 8 years, survives him.

He was a member of the American Society of Mechanical Engineers, the American Institute of Chemical Engineers, the Engineers' Society of Western Pennsylvania, the American Chemical Society, Pittsburgh Commandery No. 1, Knights Templar, Syria Temple, A. A. O. M. S., and the Duquesne Club.

Mr. Greth was elected a Member of the American Society of Civil Engineers on March 4th, 1913.

WILLIAM MACKENZIE HUGHES, M. Am. Soc. C. E.*

Died June 25th, 1915.

William Mackenzie Hughes was born in Utica, N. Y., on June 5th, 1848. He began his career as a machinist's apprentice and worked at that trade until September, 1869, when he entered Cornell University. He left college, however, in February, 1871, without taking his degree.

In 1871 and 1872, Mr. Hughes was employed as Assistant to Marvin Porter, Chief Engineer of the Walnut Hills Tunnel, at Cincinnati, Ohio, and during the spring and summer of 1873, he was engaged as an Architectural Draftsman and also as Assistant Engineer on the survey of the Big Sandy Valley Railroad, under Mr. Porter, as Chief Engineer.

Mr. Hughes had chosen to make bridge and structural steelwork his specialty in engineering, and in September, 1873, he entered the service of the Cincinnati Bridge Company, as Assistant on the design and erection of bridges. He remained with that Company until 1876, when he was employed on bridge construction by Mr. Charles Graham, of Cincinnati.

In October, 1876, he was appointed Assistant City Engineer of the City of Cincinnati, in charge of bridges and bridge construction, which position he held until 1881, when he entered the service of the New York, Chicago and St. Louis Railway Company.

From 1883 to 1888, Mr. Hughes served as Bridge Engineer of the City of Cleveland, Ohio, and during 1889 and 1890, he was engaged as Engineer and Assistant General Manager with the Keystone Bridge Company. He left this Company in February, 1891, to become Engineer of Construction at the Columbian Exposition, under the late

* Memoir prepared by the Secretary from information on file at the House of the Society.

Abraham Gottlieb, M. Am. Soc. C. E., but resigned that position in August of the same year.

During 1893 and 1894, Mr. Hughes was Bridge Engineer of the City of Chicago, and from 1892 to 1896, during the construction of the Metropolitan (West Side) Elevated Railroad, he served that Company as Consulting Engineer. In 1896, he was appointed Bridge Engineer of the Sanitary District of Chicago, and remained in that position until May 1st, 1898, when he resigned to engage in private practice as Consulting Engineer, continuing in that capacity until his death. He is survived by his widow and one daughter.

Those who knew Mr. Hughes prized his friendship and appreciated the force of character back of his quiet, unobtrusive manner. By his death, the Engineering Profession has lost a useful member and his associates a valued friend.

Mr. Hughes was a member of the Engineers' Club of Chicago, and was elected a Member of the American Society of Civil Engineers on June 2d, 1880.

CHARLES EZEKIEL RASINSKY, Assoc. M. Am. Soc. C. E.*

Died December 26th, 1914.

Charles Ezekiel Rasinsky was born at Mohileff, Russia, on March 24th, 1865. His early life was spent in his native town, and there he acquired his primary education and was graduated from the local high school. His parents were well to do, and he was desirous of attending the University, but was unable to do so on account of the restrictions placed on the number of Jewish students admitted. Partly on this account and partly because of his liberal views, he decided to come to America, which he did in 1883 without his parents' consent. He supported himself in several lines of work for three years at various places; coming to Cincinnati, Ohio, in 1886, he entered the University for the Civil Engineering course. He worked his way through college and was graduated in 1890 with distinction. Both during and after his college career, he was a profound student of technical literature in English, French, German, and Russian.

After his graduation Mr. Rasinsky entered the Engineering Department of Cincinnati as a Rodman. He remained in this position only a few months, becoming, in the fall and winter of 1890-91, Inspector on the filtration plant at Jeffersonville, Ind., under John W. Hill, M. Am. Soc. C. E., Chief Engineer. Early in 1891, he was appointed Assistant Engineer in the Engineering Department of Cincinnati, where he remained until 1897, his work consisting largely of street and highway construction.

* Memoir prepared by H. F. Shipley, M. Am. Soc. C. E.

During 1898 and 1899, Mr. Rasinsky was Assistant on the construction of the new Cincinnati Water-Works under the late L. G. F. Bouscaren, M. Am. Soc. C. E., Chief Engineer, being engaged especially in the detailed calculations for reservoir and heavy masonry layout, and also in the design of structural and special casting work.

In 1900, he again entered the City Engineering Department of Cincinnati and continued there until 1906. His work during this time consisted in street construction and layout, and the design and construction of retaining walls, bridge piers, and abutments. During all these years Mr. Rasinsky was doing a great deal of structural designing and detailing for local shops outside of regular office hours, and, in 1906, he entered the employ of L. Schreiber and Sons Company, as a Structural Designer and Detailer. After a few months, however, he went with the Ohio Electric Company, at Dayton, Ohio, as Bridge Engineer on the proposed reconstruction of its line from Dayton to Hamilton, Ohio. He designed all the structural and reinforced concrete work on this line, some of which, however, was not put into use for several years on account of the curtailment brought on by the financial stringency of 1907.

In 1908, he again entered the service of the City Engineering Department of Cincinnati, this time in the Sewerage Division, under J. H. Sundmaker, City Engineer. In 1910-11, when the writer became City Engineer, Mr. Rasinsky assumed general charge of the entire Sewer Division. This branch of the Engineering Department had been retrograding for many years, and had become an affair of mere routine, no work of a general nature being carried on. Mr. Rasinsky immediately began a reorganization along more advanced and modern lines, but lack of financial support prevented the carrying out of his plans on the broad scale required to achieve real results in Cincinnati.

In 1912, he entered private practice as a Civil and Consulting Engineer, and continued as such until his death. During the last two years of this time he was associated with the writer under the firm name of Shipley and Rasinsky.

Mr. Rasinsky was married in 1891 to Miss Rebecca Tuttleman, who, with three children, survives him.

He was a man of sterling character and of very considerable scientific attainments, especially in the higher mathematics. He was a pioneer local designer in reinforced concrete, and was versed in all the literature of this subject in four languages. He never reached the higher ranks in the engineering world, but he was a man who matured late in life, and was apparently ready for his best work at the time of his sudden death.

Mr. Rasinsky was elected an Associate Member of the American Society of Civil Engineers on July 9th, 1912.

PAPERS IN THIS NUMBER

"THE HYDRAULIC JUMP, IN OPEN-CHANNEL FLOW AT HIGH VELOCITY." KARL R. KENNISON. (To be presented Nov. 3d, 1915.)

"A STUDY OF THE DEPTH OF ANNUAL EVAPORATION FROM LAKE CONCHOS, MEXICO." EDWIN DURYEA, JR., and H. L. HAEHL. (To be presented Nov. 17th, 1915.)

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The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

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* Vacancy in chairmanship caused by the death of Austin Lord Bowman.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

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MINUTES OF MEETINGS

OF THE SOCIETY

September 15th, 1915.—The meeting was called to order at 8.30 P. M.; T. Kennard Thomson, M. Am. Soc. C. E., in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 85 members and 10 guests.

A paper by J. B. T. Colman, Assoc. M. Am. Soc. C. E., entitled "The Action of Water Under Dams", was presented by the Assistant Secretary, who also read communications on the subject from Messrs. H. B. Muckleston, Malcolm Elliott, and Warren B. Travell. The paper was discussed by Messrs. Edward Wegmann and W. P. Creager.

A paper by E. D. Cole, Assoc. M. Am. Soc. C. E., entitled "Concrete-Lined Oil-Storage Reservoirs in California: Construction Methods and Cost Data", was presented by the Assistant Secretary.

The Assistant Secretary announced the following deaths:

John William Eber, of Hamilton, Ont., Canada, elected Associate Member, September 4th, 1907; Member, September 6th, 1910; died August 18th, 1915.

Gunardo Anfin Lange, of Mendoza, Argentine Republic, elected Member, March 3d, 1897; died July 24th, 1915.

Adjourned.

October 6th, 1915.—The meeting was called to order at 8.30 P. M.; J. Waldo Smith, M. Am. Soc. C. E., in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 242 members and 56 guests.

The minutes of the meeting of September 1st, 1915, were approved as printed in September, 1915, *Proceedings*.

A paper by John Vipond Davies, M. Am. Soc. C. E., entitled "The Astoria Tunnel Under the East River for Gas Distribution in New York City", was presented by the author and illustrated with lantern slides.

The paper was discussed by Messrs. W. H. Bradley, Edward Wegmann, Hugh L. Cooper, Thomas H. Wiggin, T. Kennard Thomson, R. C. Kellogg, James F. Sanborn, W. J. Boucher, W. W. Brush, C. R. Hulsart, and the author. The Assistant Secretary announced that he had received written communications on the subject from Messrs. F. Lavis and Milton H. Freeman, but these were not read owing to the lateness of the hour.

The Assistant Secretary announced the following deaths:

Lindsey Louin Jewel, of Saranac Lake, N. Y., elected Associate Member, April 4th, 1906; Member, November 1st, 1910; died September 5th, 1915.

Maxymilian Lewinson, of New York City, elected Member, February 7th, 1894; died September 19th, 1915.

William Watson, of Boston, Mass., elected Associate, March 1st, 1882; Member, September 2d, 1891; died September 30th, 1915.

Adjourned.

FORTY-SEVENTH ANNUAL CONVENTION

HELD IN SAN FRANCISCO, CAL., SEPTEMBER 16TH–18TH, 1915

FIRST SESSION

Thursday, September 16th, 1915.—The first session of the Convention was opened in the St. Francis Hotel at 10 A. M.; Charles D. Marx, President, Am. Soc. C. E. in the chair; Chas. Warren Hunt, Secretary; and present, also, about 400 members and guests.

The President delivered the Annual Address.*

Adjourned.

SECOND SESSION, BUSINESS MEETING †

Thursday, September 16th, 1915.—Immediately after the President had delivered the Annual Address, the Business Meeting was called to order.

The Secretary presented a report on the suggestions of members as to the Time and Place for holding the Annual Convention of 1916.

The Secretary read a telegram from a number of members inviting the Society to hold its next Annual Convention in Pittsburgh, Pa., and announced that he had received a number of letters from Mayors of Cities, Chambers of Commerce, etc., containing similar invitations.

On motion, duly seconded, it was ordered that the matter of the Time and Place for holding the next Annual Convention be referred to the Board of Direction with power.

The Secretary announced the result of the canvass of ballots on the question of rescinding the following resolution adopted at the Annual Meeting on January 18th, 1911, as follows: "That it is the sense of this meeting that the licensing of Engineers by States is undesirable". The ballot was canvassed on August 31st, 1915, and resulted in 2 024 ballots in favor of rescinding and 592 against, so that the resolution was rescinded in accordance with the recommendation of the Board of Direction.

The Secretary announced that at a meeting of the Seattle Association of Members of the American Society of Civil Engineers held June 28th, 1915, the following report was received from a special committee, consisting of Messrs. Bertram D. Dean and Robert Howes, appointed to review the paper entitled "Suggested Changes and Extension of the United States Weather Bureau Service in California", and that the recommendations therein made were adopted by that Association:

"The paper by Messrs. Binckley and Lee in the February issue of the *Proceedings* of the Am. Soc. C. E., 1915, appears to consider the

* The Presidential Address is printed in this number of *Proceedings*.

† The Report in Full of the Business Meeting will be printed in the *Proceedings* for November, 1915.

United States Weather Bureau as having for its primary object the collection of information relative to water supply, and makes sundry criticisms and recommendations.

"Your committee finds that, at least in our district, the Weather Bureau has a large sphere of usefulness in other directions, which must be taken into consideration in locating many of their stations, and we hesitate to criticize the location of existing stations. We are, however, impressed that the development of the State would be materially assisted by extending the observations to supply additional information regarding precipitation, evaporation, and snow storage, in the mountainous territory, which is the chief source of our water supply.

"We recommend that this Association request the parent Society to take definite steps to encourage the establishment of additional stations, and collect information relative to precipitation, evaporation, and snow storage in the mountains of Washington and Oregon as well as California; also to recommend that provision be made for more frequent inspection of stations by the Department's trained field observers."

On motion, duly seconded, the resolution was referred to the Board of Direction.

E. J. Schneider, M. Am. Soc. C. E., of the Local Committee, made a number of announcements in reference to excursions and entertainments.

Adjourned.

MINUTES OF MEETINGS OF THE BOARD OF DIRECTION

SAN FRANCISCO, CAL., SEPTEMBER 16TH, SEPTEMBER 20TH, AND SEPTEMBER 21ST, 1915

(Abstract)

September 16th, 1915.—1 P. M.—The Board met, in accordance with the Constitution, at the Annual Convention, St. Francis Hotel, San Francisco, Cal.; President Marx in the chair; Chas. Warren Hunt, Secretary; and, present, also, Messrs. Bensel, Connor, Crocker, Fuller, Haskell, Hedges, Loweth, and Williams.

Adjourned.

September 20th, 1915.—2.20 P. M.—A regular meeting of the Board was held at the St. Francis Hotel, San Francisco, Cal.; President Marx in the chair; Chas. Warren Hunt, Secretary; and, present, also, Messrs. Connor, Crocker, Fuller, Haskell, Hedges, Loweth, Swain, and Williams.

The report of the tellers on Membership Ballot canvassed August 31st, 1915, was received, and the President declared the election of 12 Members, 87 Associate Members, 2 Associates, 30 Juniors, and the transfer of 18 Juniors to the grade of Associate Member, and 1 Junior to the grade of Associate.

The Secretary reported that the work of repairing the Society House has been in progress this Summer, that the whole House has been put in good order and has been redecorated.

The appointment of Messrs. Alfred Craven and A. M. Hunt by this Society, at the request of the Hon. Josephus Daniels, Secretary of the Navy, to act on a Naval Advisory Board, was reported.

The Constitution of the San Diego Association of Members of the American Society of Civil Engineers was approved.

Mr. Hunt reported the present state of the work of the Alfred Noble Memorial Committee.

Adjourned.

September 21st, 1915.—The Board reconvened at the St. Francis Hotel, San Francisco, Cal., at 10 P. M.; President Marx in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Connor, Crocker, Haskell, Hawley, Leonard, Loweth, Metcalf, and Williams.

A report was received from the Membership Committee which had held two meetings since the adjournment of the Board.

Four Associate Members were transferred to the grade of Member.

Applications were considered and other routine business was transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

November 3d, 1915.—8.30 P. M.—A regular business meeting will be held, and a paper by Karl R. Kennison, Assoc. M. Am. Soc. C. E., entitled, "The Hydraulic Jump, in Open-Channel Flow at High Velocity", will be presented for discussion.

This paper was printed in *Proceedings* for September, 1915.

November 17th, 1915.—8.30 P. M.—At this meeting a paper by Edwin Duryea, Jr., M. Am. Soc. C. E., and H. L. Haehl, Assoc. M. Am. Soc. C. E., entitled, "A Study of the Depth of Annual Evaporation from Lake Conchos, Mexico", will be presented for discussion.

This paper was printed in *Proceedings* for September, 1915.

December 1st, 1915.—8.30 P. M.—This will be a regular business meeting. Two papers will be presented for discussion, as follows: "The Automatic Volumeter", by E. G. Hopson, M. Am. Soc. C. E.; and "The Cherry Street Bridge, Toledo, Ohio", by Clement E. Chase, Jun. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

LIST OF NOMINEES FOR THE OFFICES TO BE FILLED AT THE ANNUAL ELECTION, JANUARY 19th, 1916.

The following list of nominees for the offices to be filled at the Annual Meeting, January 19th, 1916, received from the Nominating Committee, was presented to the Board of Direction at its meeting on May 5th, 1915. The list has already been mailed to all Corporate Members:

For President, to serve one year:

E. L. Corthell, New York City

For Vice-Presidents, to serve two years:

Palmer C. Ricketts, Troy, N. Y.
Alfred Craven, New York City

For Treasurer, to serve one year:

Lincoln Bush, New York City

For Directors, to serve three years:

Virgil G. Bogue, New York City............District No. 1
Alexander C. Humphreys, New York City...District No. 1
Otis F. Clapp, Providence, R. I.............District No. 2
Richard Khuen, Pittsburgh, Pa.............District No. 4
F. G. Jonah, St. Louis, Mo.................District No. 5
Edwin Duryea, Jr., San Francisco, Cal......District No. 7

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In making a search it sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is small. This method is particularly useful when there are drawings or figures in the text, which would be very expensive to reproduce by hand.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings.*

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be

published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings.*

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M., every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, L. R. Hinman, 1400 West Colfax Ave., Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, at 12.30 P. M., at the Albany Hotel.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meeting)

June 25th, 1915 (Special Meeting).—The meeting was called to order at the Denver Athletic Club; President Field in the chair; L. R. Hinman, Secretary; and present, also, 29 members and 47 guests who were local members of the American Society of Mechanical Engineers, the American Institute of Electrical Engineers, the American Institute of Mining Engineers, and the Colorado Scientific Society.

Mr. Thomas L. Wilkinson, representing the American Society of Mechanical Engineers, and Mr. H. C. Parmalee, representing the Colorado Scientific Society, addressed the meeting briefly. They were followed by Charles Warren Hunt, Secretary, and Charles D. Marx, President, of the Society, who were the guests of honor of the Asso-

ciation. The subject of President Marx's address was "Recent Water Legislation in California."

Adjourned.

In accordance with a special programme prepared for the entertainment of President Marx and Secretary Hunt, of the Society, who had stopped in Denver for the day, *en route* from San Francisco to New York, a Reception Committee, consisting of Messrs. H. S. Crocker, E. F. Vincent, and A. O. Ridgway, met the guests at the Union Depot and escorted them to the Denver Athletic Club where, through the courtesy of Mr. Crocker, the privileges of the Club were extended to them.

An inspection of the Colfax-Larimer Viaduct was made, after which 21 members and 6 guests joined the party. Automobiles were taken to the Lakewood Golf Club where luncheon was served, after which the party took the trip to Golden, up Lookout Mountain, thence to Evergreen, returning to Denver by way of Bear Creek.

Atlanta Association

The Atlanta Association of Members of the American Society of Civil Engineers was organized on March 14th, 1912. The Association holds its meetings at the University Club, Atlanta, Ga.

At the meeting of the Association on January 9th, 1915, the following officers were elected for the ensuing year: President, Park A. Dallis; First Vice-President, B. M. Hall; Second Vice-President, P. H. Norcross; Secretary-Treasurer, T. B. Branch.

Baltimore Association

On May 6th, 1914, the Baltimore Association of Members of the American Society of Civil Engineers was organized, a Constitution adopted, and the following officers were elected: J. E. Greiner, President; Francis Lee Stuart, First Vice-President; L. H. Beach, Second Vice-President; Harry D. Williar, Jr., Secretary-Treasurer; and Messrs. H. D. Bush, B. T. Fendall, B. P. Harrison, Calvin W. Hendrick, Oscar F. Lackey, M. A. Long, and A. A. Thompson, Directors.

At its meeting of September 2d, 1914, the Board of Direction considered and approved the proposed Constitution of the Baltimore Association of Members of the American Society of Civil Engineers.

Cleveland Association

The proposed Constitution of the Cleveland Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on January 6th, 1915.

The following officers have been elected: President, Willard Beahan; Vice-President, Robert Hoffmann; Secretary-Treasurer, George H. Tinker.

Louisiana Association

At the meeting of the Louisiana Association of Members of the American Society of Civil Engineers (New Orleans, La.), on April 14th, 1915, the following officers were elected for the ensuing year: J. F. Coleman, President; W. B. Gregory and A. M. Shaw, Vice-Presidents; Ole K. Olsen, Treasurer; and E. H. Coleman, Secretary.

Northwestern Association

The proposed Constitution of the Northwestern Association of Members of the American Society of Civil Engineers (St. Paul and Minneapolis, Minn.) was considered and approved by the Board of Direction of the Society on November 4th, 1914. F. W. Cappelen is President and R. D. Thomas, Secretary.

Philadelphia Association

The meetings of the Philadelphia Association of Members of the American Society of Civil Engineers are held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

The officers of the Association are as follows: President, Richard L. Humphrey; Vice-Presidents, F. Herbert Snow and Edgar Marburg; Directors, John Sterling Deans, J. W. Ledoux, H. H. Quimby, and H. S. Smith; Treasurer, S. M. Swaab; and Secretary, W. L. Stevenson.

Portland, Ore., Association

At the meeting of the Association on October 21st, 1914, the following officers were elected for the ensuing year: President, George C. Mason; First Vice-President, W. S. Turner; Second Vice-President, John T. Whistler; Treasurer, G. B. Hegardt; and Secretary, Charles J. McGonigle.

St. Louis Association

The proposed Constitution of the St. Louis Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on October 7th, 1914.

The following officers have been elected: President, J. A. Ockerson; First Vice-President, Edward E. Wall; Second Vice-President, F. J. Jonah; Secretary-Treasurer, Gurdon G. Black. The meetings of the Association are held at the Engineers' Club Auditorium.

San Diego Association

The San Diego Association of Members of the American Society of Civil Engineers was organized on February 5th, 1915, and officers have been elected, as follows: President, George Butler; Vice-President, Willis J. Dean; and Secretary-Treasurer, J. R. Comly.

At its meeting of September 20th, 1915, the Board of Direction considered and approved the proposed Constitution of the San Diego Association of Members of the American Society of Civil Engineers.

Seattle Association

The Seattle Association of Members of the American Society of Civil Engineers was organized on June 30th, 1913. At its meeting of January 25th, 1915, the following officers were elected for the ensuing year: President R. H. Ober; Vice-President, A. S. Downey; and Secretary-Treasurer, Carl H. Reeves.

(Abstract of Minutes of Meeting)

August 30th, 1915.—The meeting was called to order at 12.15 P. M., at the College Club; Mr. John L. Hall in the chair; Carl H. Reeves, Secretary; and present, also, 14 members and guests.

The minutes of the adjourned meeting of August 2d, 1915, were read and approved.

The Committee appointed to revise the Articles of Association of The Associated Engineering Societies of Seattle, presented its report, the new Articles being read by Mr. Robert Howes, who stated that the revision was made at a meeting of the entire Committee.

Mr. Joseph Jacobs presented the following resolution:

"*Resolved:* That the Seattle Association of Members of the American Society of Civil Engineers do approve The Associated Engineering Societies of Seattle outlined in The Preliminary Articles of Agreement, Constitution and By-Laws as modified and presented by its Conference Committee; and

"*Be it Further Resolved:* That the President appoint two representatives, preferably members of our present Conference Committee, with power to co-operate with like representatives of the other societies as provided in said Preliminary Articles of Agreement."

The subject was discussed by Messrs. Carl H. Reeves, Henry L. Gray, J. A. Jackson, and Bertram D. Dean, after which, on motion, duly seconded, the resolution was adopted.

On motion, duly seconded, the request of the Special Committee on Creosoted Timbers for the payment of $5 by the Association, toward the organization expenses of the Committee, was granted.

Adjourned.

Southern California Association

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bimonthly meetings, with banquet, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary of the Association, W. K. Barnard, 514 Central Building, Los Angeles, Cal.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

Spokane Association

The proposed Constitution of the Spokane Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on March 4th, 1914. Ulysses B. Hough is President.

Texas Association

The proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on December 31st, 1913. The headquarters of the Association is Dallas, Tex. John B. Hawley is President.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria. Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 176 Mansfield Street, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark

Detroit Engineering Society, 46 Grand River Avenue, West, Detroit, Mich.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, 6 West Eager Street, Baltimore. Md.

Engineers' Club of Kansas City, E. B. Murray, Secretary, 920 Walnut Street, Kansas City, Mo.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Club of Trenton, Trent Theatre Building, 12 North Warren Street, Trenton, N. J.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, The Minories, Tower Hill, London, E., England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, State Museum Building. Chartres and St. Ann Streets, New Orleans, La.

Memphis Engineers' Club, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Oregon Society of Civil Engineers, Portland, Ore.

Pacific Northwest Society of Engineers, 312 Central Building. Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen. Denmark.

Western Society of Engineers, 1737 Monadnock Block. Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From September 2d, to October 2d, 1915)

DONATIONS*

HISTORY OF THE PANAMA CANAL,

Its Construction and Builders. By Ira E. Bennett. Associate Editors, John Hays Hammond, Patrick J. Lennox, William Joseph Showalter, Philip Andrews, Rupert Blue, J. Hampton Moore. Builders' Edition. Cloth, $11\frac{1}{2}$ x 8 in., illus., 11 + 543 pp. Washington, Historical Publishing Company, 1915. $5.00.

The preface states that an effort has been made in this book to tell the plain story of Panama from the time of its discovery, as well as that of the history and construction of the Panama Canal. Chapters I to XIII contain a concise and detailed history of Panama, and Chapters XIV to XXXIV relate to the building of the Panama Railroad and the history of the Panama Canal from its beginning by the French to its completion and operation by the American Government. Chapters XXXV to XLIX have been written by men who have had to do with the Canal since its inception as an American enterprise, namely, Theodore Roosevelt, John F. Wallace, Theodore P. Shonts, John F. Stevens, etc., etc., and therefore should be especially interesting. The text is fully illustrated and, among other interesting items, the latter chapters contain a description of the Republic of Panama and its industries; a résumé of the effects on trade, ports, and transportation by the Canal; a description of American industries represented in the building of the Canal; a list of American firms and corporations which supplied equipment, etc., for the Canal; a biographical section devoted to men who were connected with the Canal in various capacities; texts of treaties between the United States and foreign countries relating to interoceanic communications; and laws of the United States relating to the operation, maintenance, and government of the Canal, toll rates, etc. There is an Index of twenty-three pages.

MOTION OF LIQUIDS.

By R. de Villamil. Cloth, $8\frac{3}{4}$ x $5\frac{3}{4}$ in., illus., 14 + 210 pp. London, E. & F. N. Spon, Limited; New York, Spon & Chamberlain, 1914. $2.50.

Having treated of the fundamentals of the subject, with particular reference to the assumptions on which the mathematical treatment has been based, in a previous volume, the "A. B. C. of Hydrodynamics", the author, in the present volume, has developed the subject, it is said, by practical application to definite cases of resistance, especially examining and studying the experimental work carried out in this connection by Dubuat and Duchemin. Starting with the assumption that when a body moves in a liquid, the latter moves, by some means or other, from the front to the rear of the body, the author, it is stated, has endeavored to show, step by step, and in as simple a manner as possible, how the liquid moves by reference to various suitable experiments, and by confining his attention chiefly to flat plates, he has been able to treat the fluid as if it were inviscid. Among the subjects especially discussed by the author are the difference between a static and a non-static liquid, the importance of the "coefficient of contraction" in the treatment of the passage of a liquid through a hole in a thin partition, whether a body floating in a stream moves faster than the stream, and the idea of measuring the resistance of a liquid by the amount of energy wasted, which the author calls "negative resistance", etc., etc. At the end of each chapter is a Summary of the matter discussed in that chapter and a brief bibliography of the subject. The Contents are: Introductory Remarks; The "Divide"; Motion of Liquid Round the Plate, etc.; Relative Motion; Motion of the Liquid at the Side of the Plate; Water Flowing in Jets; Jets Striking a Plate at an Angle; Explanation of "Dubuat's Paradox"; Movement of Liquids Through Apertures in the Wall of a Vessel, etc.; Rivers and Canals; Negative Resistance in Liquids; Curves of Resistance; Index.

IRRIGATION PRACTICE AND ENGINEERING:

Volume I, Use of Irrigation Water and Irrigation Practice. By B. A. Etcheverry, Assoc. M. Am. Soc. C. E. Cloth, $9\frac{1}{2}$ x $6\frac{1}{4}$ in., illus., 13 + 213 pp. New York and London, McGraw-Hill Book Company, Inc., 1915. $2.00.

This volume is the first of three which, it is stated, the author hopes will fill the needs of teachers and students in technical colleges and universities and which may

* Unless otherwise specified, books in this list have been donated by the publishers.

be used as a reference book by engineers engaged in irrigation work, as well as by managers and superintendents of irrigation systems. In this, the introductory volume, Chapters I to V relate to the use and disposal of water on irrigation systems, and Chapter VI describes the methods of preparing land for irrigation and of applying the water. In Chapter VII, the author describes ditches and structures for the conveyance and distribution of water on farms, and in Chapter VIII, he deals with small pumping plants, including descriptions of various types of installation, conditions for which each type is best adapted, cost of installation, annual cost of operation and maintenance, etc. In discussing the subject, the author, it is said, has confined himself largely to irrigation practice in the United States, and has illustrated his subject with carefully selected examples, which, in many cases, are accompanied by cost data, etc. At the end of each chapter is a brief bibliography to which the reader is referred for more detailed information. The Contents are: Soil Moisture and Plant Growth, and Their Bearing on Irrigation Practice; Disposal of Irrigation Water Applied to the Soil, Plant Transpiration, Soil Moisture Evaporation, Soil Water Percolation, Surface Waste; Water Requirement of Irrigated Crops; Results of Investigations and Irrigation Practice Regarding Proper Time to Irrigate, Frequency of Irrigations for Different Crops, Irrigation Season; Duty of Water; Preparation of Land for Irrigation and Method of Applying Water to the Land; Farm Ditches and Structures for the Distribution of Irrigation Water; The Selection and Cost of a Small Pumping Plant; Index.

MODERN PLUMBING ILLUSTRATED:

A Comprehensive and Thoroughly Practical Work on the Modern and Most Approved Methods of Plumbing Construction. By R. M. Starbuck. Third Edition, Revised and Enlarged. Cloth, 10½ x 7½ in., illus., 407 + 30 pp. New York, The Norman W. Henley Publishing Company, 1915. $4.00.

On the title-page it is stated that this book is a standard work for plumbers, architects, builders, property owners, boards of health and plumbing examiners, and for trade classes in plumbing. The author, it is said, has covered the entire field of plumbing, not only as practiced in towns and cities, under strict regulations, but also in country districts where conditions are entirely different. Fixture work, the construction of the drainage and vent systems, and of complete plumbing systems of buildings of various kinds, together with suggestions for estimating plumbing construction, have been discussed and described in detail. The subject of water supply, especially in country districts and in large city buildings is closely associated, it is stated, with drainage problems, and although this book is designed to cover subjects pertaining to drainage alone, the author has deemed it advisable to discuss the general subject of water supply in these connections. It is hoped that the book which deals entirely with modern methods and modern appliances will prove of interest and help to those who have to do with the betterment of plumbing construction and of the plumbing trade at large. A partial list of Contents is: The Kitchen Sink, Laundry Tubs; Lavatories; Baths; Water Closet Connections; The Low-Down Water Closet; The Slop Sink; The Hotel or Restaurant Sink; Refrigerators; Refrigerator Lines; The Stall Sink; Connections for S-Traps; Connections for Drum Traps; Soil Pipe and Soil Pipe Connections; Supporting and Running of Soil Pipe; The House or Main Trap and Fresh Air Inlet; Floor and Yard Drains; Water Closets; Local Venting; Bath Rooms; Poor Practices in Plumbing Construction; "Roughing-In"; Testing of the Plumbing System; Continuous Venting; Plumbing for Cottage House; Construction of Cellar Piping; Plumbing for Residences; Plumbing for Two-Flat House; Plumbing for Apartment Building; Plumbing for Double Apartment Buildings; Plumbing for Office Buildings; etc., etc. Each chapter is illustrated with a full-page plate, and there is an index of nineteen pages.

LOCATION OF CARBURETION SYSTEM TROUBLES MADE EASY.

Chart Arranged by Victor W. Page. Paper, 24 x 38 in. New York, Norman W. Henley Publishing Company, 1915. 25 cents.

As stated in a secondary title, this chart contains in outline "a complete concise exposition showing common derangements that are apt to occur in the carburetion group of automobile power plants that will interfere with smooth engine action", and is intended as a ready reference guide for locating faults in the carburetion system. The carburetor and fuel supply system of a typical automobile power plant are shown in cross-section, with all parts of the fuel supply group so clearly illustrated that, it is stated, even the novice motorist will recognize them, and causes of trouble, as well as how to locate defects and means of remedying them, are discussed briefly.

SURVEYING MANUAL:

A Manual of Field and Office Methods for the Use of Students in Surveying. By William D. Pence and Milo S. Ketchum, Members, Am. Soc. C. E. Fourth Edition. Roan, 7 x $4\frac{1}{2}$ in., illus., 15 + 388 pp. New York, McGraw-Hill Book Company, Inc., 1915. $2.00.

The first edition of this book was issued in 1900. For this, the fourth edition, the subject-matter, it is said, has been critically revised and many changes and additions have been made to increase its usefulness. Nearly all the cuts have been re-drawn, it is stated, and numerous tables have been added. The authors' aim, as stated in the preface, has been to give clear, definite, and concise descriptions of surveying instruments and surveying methods, instructions in drawing and freehand lettering, methods of computing, field practice, etc., in order to familiarize the student with the approved surveying methods and cultivate his skill in keeping good field notes and making reliable calculations. Practice problems in the subject discussed in that chapter are given at the end of each chapter, and numerous samples of field notes and other working forms are included in the text. The Chapter Headings are: General Instructions; The Chain and Tape; The Compass; The Level; The Transit; Topographic Surveying; Land Surveying; Railroad Surveying; Errors of Surveying; Methods of Computing; Topographic Drawing and Freehand Lettering; Field and Office Tables; Index.

Gifts have also been received from the following:

Am. Electrochemical Soc. 1 vol.
Am. Inst. of Chemical Engrs. 1 bound vol.
Am. Inst. of Metals. 1 vol.
Am. Inst. of Min. Engrs. 1 vol.
Am. Road Builders' Assoc. 1 vol.
Am. Soc. of Mech. Engrs. 1 bound vol.
Am. Telephone & Telegraph Co. 1 vol.
Assoc. of Ry. Telegraph Supts. 1 pam.
Associazione Elettrotecnica Italiana. 1 pam.
Atlantic Deeper Waterways Assoc. 1 pam.
Beckstrand, E. H. 1 pam.
Berlin Kgl. Technische Hochschule. 2 pam.
Binnie, W. J. E. 1 pam.
Boston, Mass.-Directors of the Port of Boston. 1 pam.
Canada-Dept. of Mines. 1 vol.
Canada-Geol. Survey. 2 vol., 5 pam.
Canada-Rys. Land Branch. 2 maps.
Canadian Engineer, The. 1 pam.
Carnegie Endowment for Inter. Peace. 1 bound vol.
Chemin de Fer de Paris á Orleans. 4 pam.
Chicago & North Western Ry. Co. 1 pam.
Chicago, Indianapolis & Louisville Ry. Co. 1 pam.
Chicago, Milwaukee & St. Paul Ry. Co. 1 pam.
Chicago, St. Paul, Minneapolis & Omaha Ry. Co. 1 pam.
City Managers' Assoc. 2 pam.
Coggeshall, R. C. P. 1 pam.
Colombia-Secretario de Relaciones Exteriores. 1 pam.
Conference of Mayors and Other City Officials of the State of New York. 4 vol.
Council Bluffs, Iowa-Water Dept. 1 vol.
Crehore, W. W. 10 vol.
Davenport, Iowa-Bldg. Insp. 1 pam.
Denver & Rio Grande R. R. Co. 1 pam.
Dist. of Columbia-Public Utilities Comm. 1 pam.
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Theatres: Their Safety from Fire and Panic, Their Comfort and Healthfulness. By William Paul Gerhard. Second Edition. New York, 1915.

Handbuch des Wasserbaues für das Studium und Praxis. 2 Vol. Leipzig and Berlin, 1914.

Hand Book of Natural Gas. By Henry P. Westcott. Erie, Pa., 1913.

The Mineral Industry: Its Statistics, Technology and Trade During 1914. Edited by G. A. Roush. Vol. 23. New York and London, 1915.

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The Economics of Contracting: A Treatise for Contractors, Engineers, Manufacturers, Superintendents and Foremen Engaged in Engineering Contracting Work. Vol. 2. By Daniel J. Haner. Chicago, 1915.

Poor's Manual of Industrials, Manufacturing, Mining and Miscellaneous Companies, 1915. Sixth Annual Number. New York.

English Local Government: The Story of the King's Highway. By Sidney and Beatrice Webb. New York and London, 1913.

Report of the Commissioner of Corporations on the Transportation of Petroleum, May 2d, 1906. Washington.

The Diesel or Slow-Combustion Oil Engine: A Practical Treatise on the Design and Construction of the Diesel Engine for the Use of Draughtsmen, Students, and Others. By G. James Wells and A. J. Wallis-Tayler. Second Edition, Revised. New York, 1915.

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Public Utilities Reports Annotated: Containing Decisions of the Public Service Commissions and of State and Federal Courts. A. B. Rochester, N. Y., 1915.

A Bibliography of Municipal Government in the United States. By William Bennett Munro. Cambridge, Mass., and London, 1915.

The Distribution of Gas. By Walter Hole. Third Edition. New York and London, 1912.

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(From September 2d to October 2d, 1915)

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MCGREW, JOHN ALEXANDER. Supt., Delaware & Hudson Co., Albany, N. Y.		Aug. 31, 1915
MORRISON, JOSEPH S. Fifth Dist. Highway Engr., State Highway Comm., 608 Spring St., Ottumwa, Iowa		Aug. 31, 1915
SCHEIDENHELM, FREDERICK WILLIAM. Cons. Engr., Benedum Trees Bldg., Pittsburgh, Pa.	Assoc. M. M.	June 6, 1911 July 7, 1915
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BELL, HAROLD INGERSOLL. Mgr., Portland Office, and Engr., H. P. Cummings Constr. Co., 409 Fidelity Bldg., Portland, Me.		Aug. 31, 1915
BILES, GEORGE HERBERT. Second Deputy State Highway Commr., Harrisburg, Pa.		Aug. 31, 1915
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BOYLE, JAMES ALPHONSUS. Civ. and Structural Engr. (Boyle & Howe), 401 Bennett Bldg., Wilkes-Barre, Pa.		Aug. 31, 1915

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ROBINSON, WILLIAM ERNEST. 420 East Walnut St., Springfield, Mo	Aug. 31, 1915
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DEVIN, GEORGE. 78 De Menil Bldg., St. Louis, Mo.

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FOSTER, SAMUEL DAVIS. Cons. Highway Engr., Room 1518, Farmers Bank Bldg., Pittsburgh, Pa.

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DEATHS

EBER, JOHN WILLIAM. Elected Associate Member, September 4th, 1907; Member, September 6th, 1910; died August 18th, 1915.

JEWEL, LINDSEY LOUIN. Elected Associate Member, April 4th, 1906; Member, November 1st, 1910; died September 5th, 1915.

LANGE, GUNARDO ANFIN. Elected Member, March 3d, 1897; died July 24th, 1915.

LEWINSON, MAXYMILIAN. Elected Member, February 7th, 1894; died September 19th, 1915.

WARDER, JOHN HAINES. Elected Associate, March 7th, 1888; died August 30th, 1915.

WATSON, WILLIAM. Elected Associate, March 1st, 1882; Member, September 2d, 1891; died September 30th, 1915.

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7 886.

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(September 2d to October 2d, 1915)

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In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

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(2) *Proceedings,* Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur,* München, Germany.
(8) *Stevens Institute Indicator,* Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(11) *Engineering* (London). W. H. Wiley, 432 Fourth Ave., New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *Engineering Record,* New York City, 10c.
(15) *Railway Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway Review,* Chicago, Ill., 15c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 20c.
(21) *Railway Engineer,* London, England, 1s. 2d.
(22) *Iron and Coal Trades Review,* London, England, 6d.
(23) *Railway Gazette,* London, England, 6d.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *Railway Age Gazette,* Mechanical Edition, New York City, 20c.
(26) *Electrical Review,* London, England, 4d.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(36) *Cornell Civil Engineer,* Ithaca, N. Y.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(39) *Technisches Gemeindeblatt,* Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung,* Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift,* Berlin, Germany.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Colliery Engineer,* Scranton, Pa., 25c.
(46) *Scientific American,* New York City, 15c.
(47) *Mechanical Engineer,* Manchester, England, 3d.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia, 25 kop.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Vereln, Vienna, Austria, 70h.
(54) *Transactions,* Am. Soc. C. E., New York City, $12.
(55) *Transactions,* Am. Soc. M. E., New York City, $10.
(56) *Transactions,* Am. Inst. Min. Engrs., New York City, $6.

(57) *Colliery Guardian,* London, England, 5d.
(58) *Proceedings,* Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings,* American Water-Work Assoc., Troy, N. Y.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
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(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 5c.
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(68) *Mining Journal,* London, England, 6d.
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(90) *Transactions,* Inst. of Naval Archts., London, England.
(91) *Transactions,* Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin,* Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie,* Paris, France, 4 fr. 50.
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(99) *Proceedings,* Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs,* Corps of Engrs., U. S. A., Washington, D. C., 50c.
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(102) *Organ für die Fortschritte des Eisenbahnwesens,* Wiesbaden, Germany.
(103) *Mining and Scientific Press,* San Francisco, Cal., 10c.
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(107) *Schweizerische Bauzeitung,* Zürich, Switzerland.
(108) *Iron Tradesman,* Atlanta, Ga., 10c.
(109) *Journal,* Boston Soc. C. E., Boston, Mass., 50c.
(110) *Journal,* Am. Concrete Inst., Philadelphia, Pa., 50c.
(111) *Journal of Electricity, Power and Gas,* San Francisco, Cal., 25c.
(112) *Internationale Zeitschrift für Wasser-Versorgung,* Leipzig, Germany.
(113) *Proceedings,* Am. Wood Preservers' Assoc.. Baltimore, Md.
(114) Institution of Municipal and County Engineers, London, England, 1s. 6d.

LIST OF ARTICLES

Bridges.

Bridge Clearance Diagram.* (Am. Ry. Eng. Assoc.) (85) Vol. 16, 1915.

Rail-End Connections for Drawbridges.* A. J. Himes. (85) Vol. 16, 1915.

Report of Committee VII—on Wooden Bridges and Trestles. (Am. Ry. Eng. Assoc.) (85) Vol. 16, 1915.

Proof of an Assumption in the Theory of Concrete Beams.* Ralph E. Goodwin, Jun. Am. Soc. C. E. (54) Vol. 78, 1915.

The Design and Construction of Four Reinforced Concrete Viaducts at Fort Worth, Texas.* S. W. Bowen, M. Am. Soc. C. E. (54) Vol. 78, 1915.

The Possibilities in Bridge Construction by the Use of High-Alloy Steels.* J. A. L. Waddell, M. Am. Soc. C. E. (54) Vol. 78, 1915.

Revision of the Progress Report of Committee on Reinforced Concrete Highway Bridges and Culverts. (Am. Concrete Inst.) (110) May.

* Illustrated.

Bridges—(Continued).

The Design of Concrete Highway Bridges with Reference to Standardization.* C. B. McCullough. (110) May.
Cost of Railway Footbridges.* (12) Serial beginning Aug. 27.
Design of Rectangular Concrete Beams. Howard Harding. (55) Sept.
Direct Lift Bridges Across Louisville and Portland Canal.* J. C. Oakes, M. Am. Soc. C. E. (100) Sept.
El Puente de Espana, 1626-1914.* W. C. Bunnel, Assoc. Am. Soc. C. E. (100) Sept.
New Bridge of the Chicago, Burlington & Quincy R. R. Over the Missouri River at Kansas City, Mo.* (86) Sept. 1.
A Simple Method of Determining the Stresses in Concrete Arches Due to Temperature and Rib Shortening.* Horace R. Thayer. (86) Sept. 1.
Unique Bridge Across Panama Canal.* (96) Sept. 2.
The Construction of the Foundations of the Bear River Bridge, near Digby, Nova Scotia.* (96) Sept. 2.
Three-Mile Approach Viaduct, St. Louis Municipal Bridge.* (13) Sept. 2.
Quebec Bridge Work in 1915.* (13) Sept. 2.
Handling Steel Centers for Concrete Arches.* Harold E. Ketchum. (13) Sept. 2.
Erecting the Largest Steel Arch Bridge in Existence (East River, Hell Gate, New York).* (46) Sept. 4; (87) Sept.
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Boiler Failures and What the American Society of Mechanical Engineers is Doing to Prevent Them. E. R. Fish. (55) Sept.
On the Laws of Lubrication of Journal Bearings. M. D. Hersey. (55) Sept.
Influence of Disk Friction on Turbine Pump Design.* F. zur Nedden. (55) Sept.
Some Mechanical Features of the Hydration of Portland Cement and the Making of Concrete as Revealed by Microscopic Study.* Nathan C. Johnson. (55) Sept.
The Mechanical Handling of Coal and Ashes in the Power Plant.* Clarence Coapes Brinley. (9) Serial beginning Sept.
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How to Increase Steam Production. H. Ross Callaway. (9) Sept.
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A Season with the Cement Gun.* W. G. Caples. (100) Sept.
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Measuring Temperatures for Heat Treatment Processes. F. G. Coburn. (72) Sept. 16.
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The Application of Established Legal Principles to the Jitney. W. E. Dunn. (17) Sept. 18.
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Excavating Aggregates with Drag Line and Hauling by Motor Trucks on Indiana Concrete Road Work.* Stanley E. Bates. (86) Sept. 22.
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The Cost of Handling Material with Motor *versus* Horse-Drawn Equipment. Henry F. W. Arnold. (9) Oct.
High Pressure Distribution for Small Plants. Ralph B. Wagner. (Paper read before the Michigan Gas Assoc.) (83) Oct. 1.
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Koksöfen mit oberer Beheizung.* Oskar Simmersbach. (50) July 22.
Maschinelle Baustelleneinrichtungen.* Eugen Pilz. (78) Aug. 3.
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Eine neue Räderkasten-Schnelldrehbank.* Adolf Rosenstein. (48) Aug. 14.
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Oxygen Blast for the Blast Furnace.* J. E. Johnson. (Paper read before the Min. and Metallurgical Soc. of America.) (22) Aug. 20; (105) Sept. 1.
The Rochester Mill, Nevada.* G. W. Wood. (103) Aug. 28.
Combustion Notes on Blast Furnace Coke. Hermann A. Brassert. (62) Sept. 1.
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Flotation at the Central Mine, Broken Hill. James Hebbard. (Abstract of paper read before the Australasian Inst. of Min. Engrs.) (103) Sept. 4.
Methods of Paying for Metal Contents of Ores. Charles H. Fulton. (Abstract from *Technical Paper No. 83,* U. S. Bureau of Mines.) (103) Sept. 11.
Ore Dressing at Clausthal.* E. Mackay Heriot. (16) Sept. 11.
Aluminum Precipitation at the Mill of the Butters Divisadero Company.* E. M. Hamilton and P. H. Crawford. (103) Sept. 11.
What Is Flotation?* T. A. Rickard. (103) Sept. 11.
Efficiency of the Blast Furnace Operation. Birger F. Burman. (105) Sept. 15.
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The Development of Field Fortification in the Civil War. W. C. Johnson and E. S. Hartshorn. (100) Sept.
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The Fauna and Stratigraphy of the Kent Coalfield.* Herbert Bolton. (Paper read before the Manchester Geol. and Min. Soc.) (106) Vol. 49, Pt. 4.
Boring and Drilling on Oilfields. Paul Dvorkovitz. (106) Vol. 49, Pt. 4.
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Dewatering an Anthracite Mine.* William Z. Price. (45) Sept.
Coal Stripping in Illinois.* (45) Sept.
Anticipating Mine Fires.* J. McCrystle. (Paper read before the Panther Valley Min. Inst.) (45) Sept.
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The New Lawrence Colliery.* Louis C. Maderia. (45) Sept.
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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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THE AUTOMATIC VOLUMETER

By E. G. Hopson, M. Am. Soc. C. E.

To be Presented December 1st, 1915.

Synopsis.

This paper describes an apparatus intended to gauge the flow of fluids by the collection of a proportionate part of the flow, or its equivalent, in a small vessel where it can be readily measured at any time. This result is accomplished by the use of very small orifices for the purpose of regulating the discharge into or out of the collecting vessel, and other special arrangements, whereby the pressure head under which the discharge into the collecting vessel takes place is at all times the equivalent, or a constant ratio, of the velocity head of the liquid or gas being measured.

Practically all measurements of flow of fluids on a large scale are difficult and expensive. As soon as one gets beyond the small service meter for water or gas, very expensive apparatus is generally involved. The accuracy of any apparatus with moving parts or mechanism is, moreover, frequently open to question, due to the difficulty of constructing any such mechanical device with a constant coefficient of

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

flow. To express the latter point in other words, the usual mechanical meter does not record with equal precision at different rates of flow; thus an instrument that will record correctly at a given rate will usually record too small for lesser rates, and *vice versa.*

The importance of a cheap and reliable apparatus to record the flow of water in small irrigation laterals, for individual services, induced the writer to take up the study of developing this device. The result indicates that it may be of equal utility to city water-works systems or to industrial plants, particularly where it is desired to ascertain the duty of pumping machinery, or to determine the use or waste of water in a pipe-distributing system. Its application to all forms of flow measurement necessarily follows as a matter of course; thus, if it can be used for water, it is equally available for oil, solutions of various kinds, or even for gas or steam. All fluid flow is subject to the same law that governs velocity and its relation to head, so that the application, to any fluid, of a device dependent on this factor only is a matter of detail.

In this discussion no attempt is made to describe the application of the device for other purposes than the measurement of the flow of water, and the experiments and tests herein described only have reference to water flow and measurement thereof.

The law now requires that on Government irrigation projects payment for maintenance shall be based on measured quantities of water delivered to the individual. Under present conditions, it is practically impossible to determine quantities; only a rough guess, in which one is guided to some extent by the ditch rider's observation and notes, is possible.

For irrigation use, probably one of the most dependable devices for measuring individual service is the Dethridge meter, which is merely a paddle-wheel operating a recording counter and has the advantage of actually measuring the displacement by each revolution of the wheel. Other methods of measurement, by weirs and modules, are inaccurate, save with apparatus and attention entirely prohibitive for ordinary purposes. The Venturi meter, as recently adapted for irrigation use, appears to be the best instrument on the market, so far as scientific accuracy and freedom from accidental interruptions are concerned; but it involves elaborate recording mechanism, with detailed reduction of results, and is expensive.

The volumeter herein described operates by the velocity head of the fluid which is being measured. The velocity head is communicated by a pipe leading from the fluid being measured to the volumeter. Not only is velocity head communicated, but a very small proportion of the actual flow is diverted by this pipe into the apparatus. This proportion is so small, however, and the velocity of flow in the pipe is so insignificant, that there are no appreciable losses of head by friction in the pipe to disturb the basic plan of operation by velocity head. A return pipe from the volumeter to the stream carries away the surplus liquid created by the influent flow.

So far as the volumeter is concerned, it operates by head or pressure. It will operate under the influence of any head or pressure, no matter how derived. Although it has been termed a volumeter—inasmuch as it measures volume—it actually is a velocity meter or pressure meter, in the sense that velocity, or pressure due to velocity, is its operating force. Therefore, it is absolutely essential to its successful operation that it shall be erected in such a way as to be subject to operation by the head due to velocity alone.

The apparatus may be attached to a pipe or conduit, with communication through a Pitot tube, or at submerged orifices, or on a short pipe or box flume of varying cross-section, or at an angle or bend in a pipe or conduit, all as illustrated on Fig. 9. Other methods of attachment will doubtless occur to the reader. The methods, however, are unimportant, so far as the operation of the device is affected, so long as they are consistent and such that the head communicated to the volumeter is a constant factor of the velocity head. The latter is the essential condition.

The most difficult feature of the apparatus to work out was an arrangement whereby the head, under which the interior jet operates, is kept identical with, or is a constant factor of, the velocity head of the stream. It is necessary for the apparatus to be absolutely automatic, and its response to the most minute changes of head to be immediate and of the most delicate sensitiveness. This requirement has been met. The influent entering the upper part of the lower vessel (Fig. 1) creates a pressure at its point of entry equal to the sum of the static and velocity heads of the entering water, the frictional losses in the influent pipe and orifice being negligible, due to their great size and the very low velocities in them. The lighter

medium (liquid or gas) contained in the lower vessel is thus put under pressure. The upper vessel is connected with the flowing water in such manner that only the static head is communicated to it. The flow through the small orifice connecting the two vessels is, therefore, for all practical purposes, that due to the velocity head, or a constant factor of the velocity head, only.

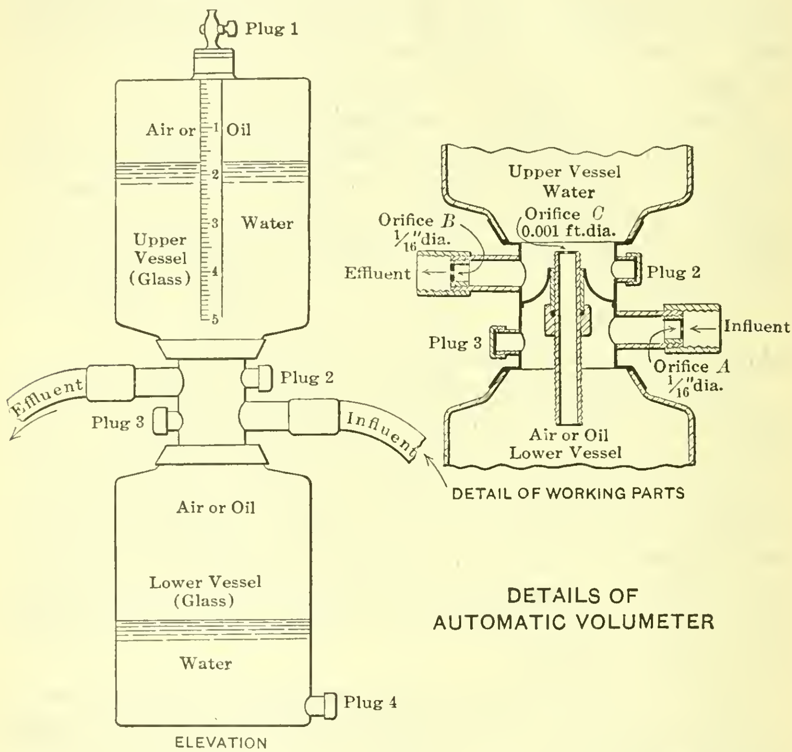


FIG. 1.

The operating details will be fully understood by reference to the detailed drawing, Fig. 1. The influent enters the lower vessel or container at its upper part through an orifice at A. The pressure communicated is due to the static head or pressure of the water being measured plus the velocity head due to the rate of flow. This pressure is communicated to the medium (air, oil, or whatever may be used) in the lower vessel, and in turn is transmitted through the

medium to the lower side of the point of connection with the upper vessel at the orifice, *C*.

The water in the effluent pipe is only under the pressure due to the static head of the stream being measured. This pressure is communicated directly through the water in the lower part of the upper vessel down to the upper side of the orifice, *C*. When there is no velocity, the static head of the stream is communicated to both the upper and lower sides of the orifice, *C*, and there is no movement. Whenever there is velocity, the corresponding head is at once communicated to the lower side of the orifice, *C*, and the balance of forces is overthrown and a movement, proportioned to $\sqrt{2g}$ multiplied by the square root of the velocity head, less whatever head is required to overcome various frictional resistances referred to later, is started.

The practical difficulties of working out this device have been considerable. Experiments to determine the coefficients of flow through such small orifices as are necessary to use revealed totally unexpected conditions, and for a time it appeared impossible to overcome these difficulties in a manner that would be satisfactory to hydraulic engineers. Experiments by the late Hamilton Smith, M. Am. Soc. C. E., and others, on the discharge through orifices of 0.02 ft. diameter and upward show coefficients of flow invariably tending to become greater with the lower velocities. This is what one would naturally expect to find, as it seems reasonable that the converging jets at the higher heads would affect the *vena contracta* more directly and forcefully than at the lower heads. Smith's experiments, however, did not go far enough for an apparatus intended for the purposes of this device.

It has been necessary to deal with flows through orifices not exceeding 0.001 to 0.004 ft. in diameter. With these small openings the ordinary laws of flow apparently do not apply with the lower heads, as can be seen by reference to the curves of coefficients, Figs. 2 to 6.

Another marked difference between results obtained with these small orifices and with the larger ones is that apparently the former have distinctly greater coefficients under certain heads than the latter. The largest coefficient noted in Smith's experiments with small orifices was about 66% of the theoretical velocity, and 90% appears to be the rule for the minute orifices referred to herein when operating under heads larger than about 0.2 ft. This is an interesting scientific fact, hard to explain, but it does not seem to be open to question.

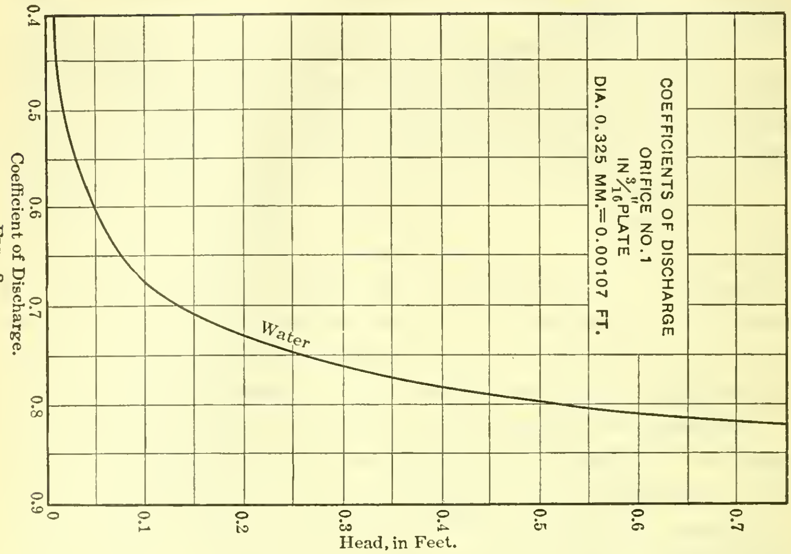

COEFFICIENTS OF DISCHARGE
ORIFICE NO. 1
IN 3/16" PLATE
DIA. 0.325 MM.=0.00107 FT.
Water
0.4
0.5
0.6
0.7
0.8
0.9
Coefficient of Discharge.
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
Head, in Feet.

FIG. 2.

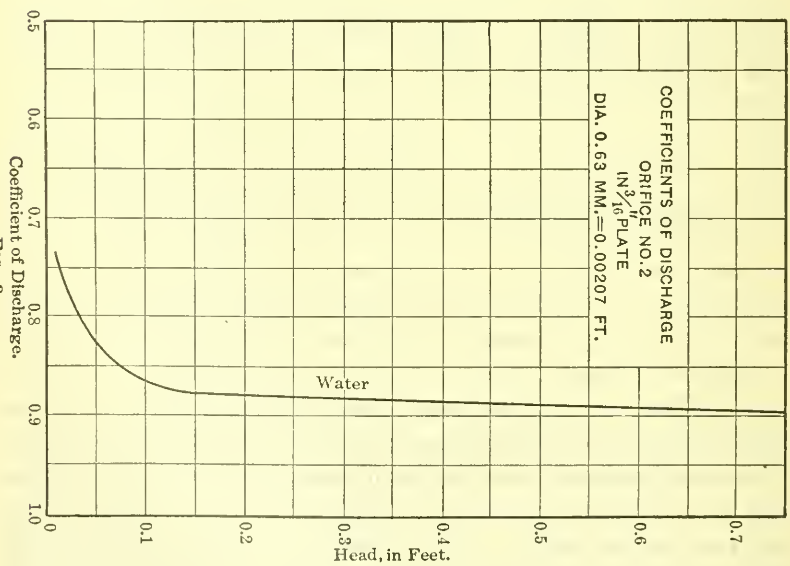

COEFFICIENTS OF DISCHARGE
ORIFICE NO. 2
IN 3/16" PLATE
DIA. 0.63 MM.=0.00207 FT.
Water
0.5
0.6
0.7
0.8
0.9
1.0
Coefficient of Discharge.
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
Head, in Feet.

FIG. 3.

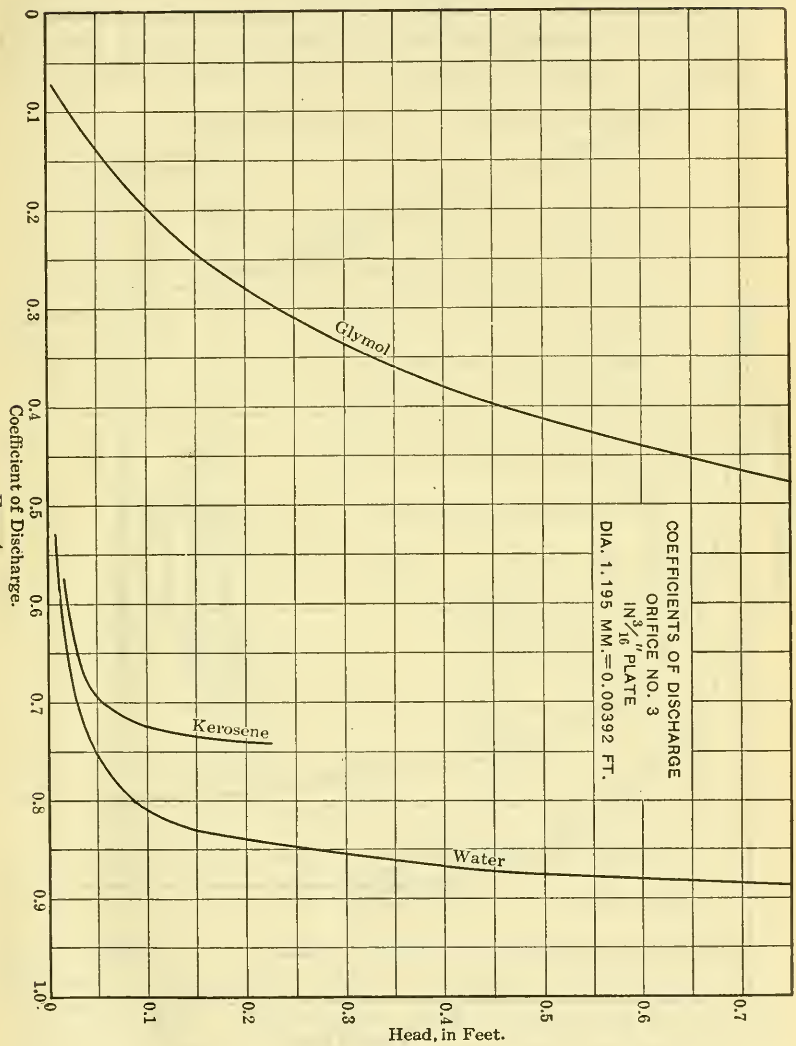
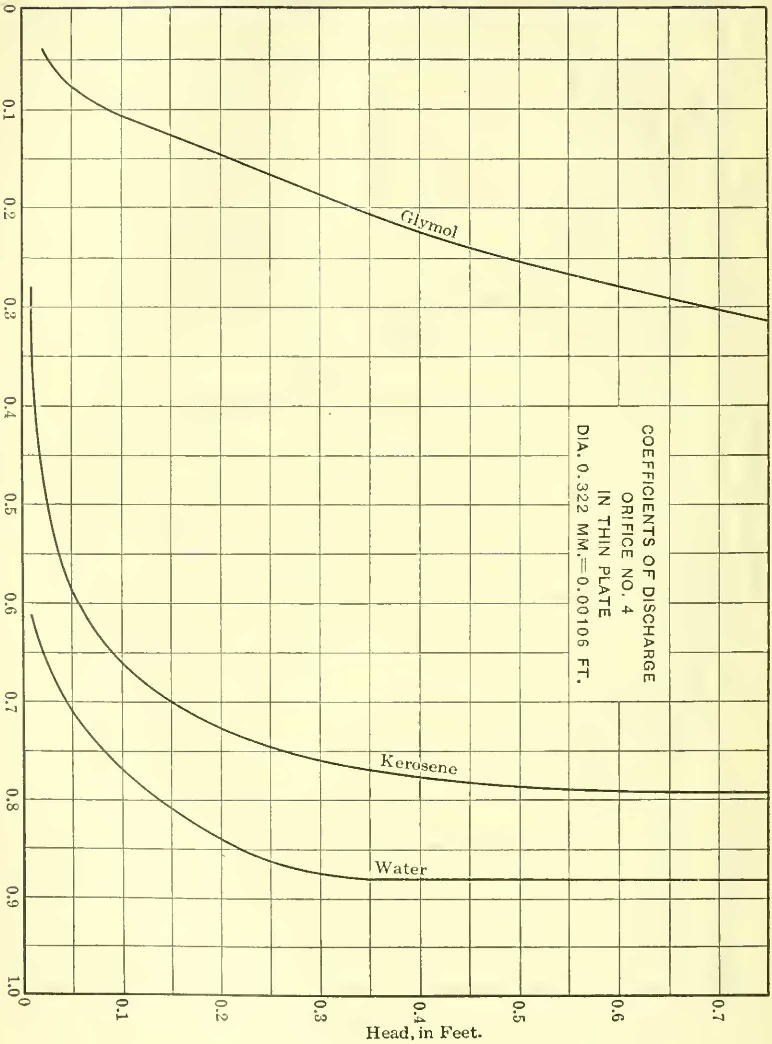

COEFFICIENTS OF DISCHARGE
ORIFICE NO. 3
IN 3/16" PLATE
DIA. 1.195 MM.=0.00392 FT.
Glymol
Kerosene
Water
Head, in Feet.
Coefficient of Discharge.
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

FIG. 4.



Coefficient of Discharge.

FIG. 5.

The curve of coefficients, Fig. 6, shows the coefficients of discharge of some of the orifices experimented on by Smith as compared with those referred to herein, and of which the largest is only about one twenty-fifth part of the size of the smallest of those used by Smith. This curve shows clearly that although the increase in coefficients of flow between the Smith orifices and these small ones is very marked,

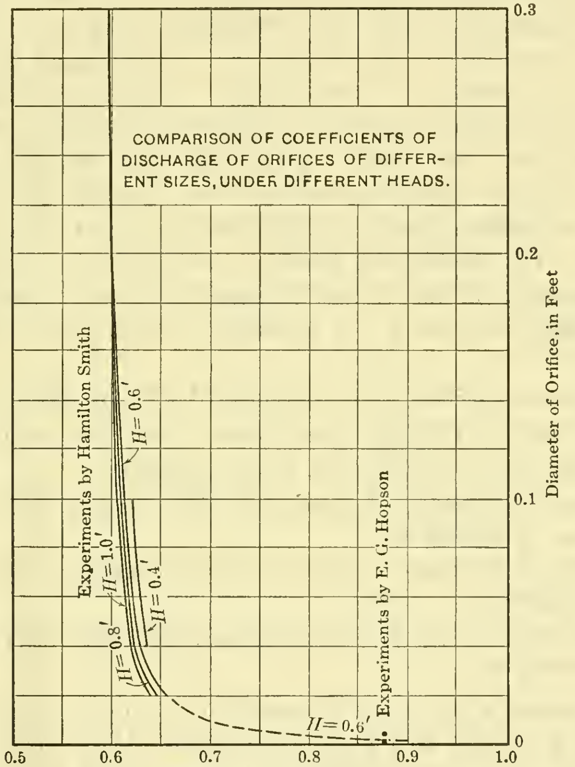


FIG. 6.

a large increase was nevertheless indicated by the sharp rise in Smith's coefficients for his smaller orifices of less than 0.2 ft. diameter.

In the conduct of experiments necessary to determine the coefficients of discharge, the diameters of orifices used were very carefully calibrated by using tapering needle points of which the diameters at different points in their length had been determined by micrometer measurements. Orifices were at first made in a brass plate, $\frac{3}{16}$-in. thick,

and, for the purpose of reducing contraction to a minimum, the upstream approach was turned—in the thickness of the metal—to an easy curve, so that the orifice itself would be in the position of the *vena contracta,* as nearly as could be determined.

In the determination of flow coefficients for these small orifices, a specially constructed apparatus was used. Water from above and below the orifice being tested was brought to two vertical glass gauge tubes, arranged side by side. The difference in level of the meniscus in each of these tubes was observed by a sliding gauge carrying a cross-hair and working on a vertical scale. In this way the difference in head could be read to less than one-thousandth of a foot. The quantity passing the orifice under a given head was carefully measured in glass graduates. These experiments were repeated a great number of times for different heads. Usually, the run lasted about an hour each time. The results were platted on cross-section paper, and the curves appearing in the various diagrams have been drawn on the basis of these tests, which, for simplicity, were omitted in the final draft.

Four different orifices were experimented with, as follows:

Orifice No. 1.—In $\frac{3}{16}$-in. plate; orifice 0.325 mm. or 0.00107 ft. diameter; up stream with curved approach.

Orifice No. 2.—Same as above, but with diameter of orifice 0.63 mm. or 0.00207 ft.

Orifice No. 3.—Same as above, but with diameter 1.195 mm. or 0.00392 ft.

Orifice No. 4.—In thin sheet metal; diameter 0.323 mm. or 0.00106 ft.

By reference to the curves of coefficients it will be found that for Orifices Nos. 2 and 3, with heads greater than 0.15 ft., the coefficients of flow ranged usually a little less than 90. With Orifice No. 1, the coefficients seemed to be appreciably less, but with Orifice No. 4, of practically the same diameter as No. 1, the only difference being that it is in thin plate, a range of coefficients almost as high as with Nos. 2 and 3 was obtained. It is apparent, however, that there is a sharper reduction in the low-head coefficients for the smaller than for the larger orifices.

With heads of 0.015 ft. and greater a fair degree of accuracy was possible with the experiments; for heads of less than this it was ex-

tremely difficult to obtain absolute accuracy. By careful repetition of experiments and extension of the curves obtained under the larger heads, it is believed, however, that the results as shown on the curves, even for the lowest heads, are very close to the truth.

The relatively small coefficients of flow under the smaller heads, which appear to be absolutely established by these experiments, necessarily introduced a disturbing element into the measuring device. It was apparent that the accuracy of the measuring device must depend on coefficients of flow for these small orifices being of reasonable constancy. The experiments, however, clearly show that for the smallest orifice the coefficients are reduced from 88% for a head of 0.3 ft. to about 60% for a head of 0.01 ft., a reduction of about 33%, an amount that would render the apparatus useless for scientific purposes.

The difficulties created by the varying coefficients of the small orifices have been overcome in the volumeter by a special feature which increases slightly the operating head, as will be referred to later.

It is an interesting fact that no advantage in flow coefficient was apparent from the curved approaches of Orifices Nos. 1, 2, and 3, in $\frac{3}{16}$-in. plate. The same, or even greater, discharges were found when the plate was reversed and the curved approach was turned down stream; moreover, Orifice No. 4, in thin plate without any curved approach, gave distinctly higher results than Orifice No. 1.

In addition to experiments with water, runs were made using oils. A careful investigation of oils on the market was made to find one having a viscosity which did not change very materially under changes of temperature such as would be encountered in a water-measuring device under working conditions (see Fig. 7 for viscosity tests). The oils most nearly approaching these conditions were kerosene, a petroleum product known commercially as white neutral oil, and another petroleum product known as glymol. The latter is a thick, clear oil, used generally for medicinal or surgical purposes. It has a viscosity ranging from about two to five times that of water or kerosene, within a range of temperatures between 32° and 100° Fahr. A series of careful runs was made with glymol and kerosene, the results being shown on the accompanying diagrams. It will be noted that the coefficients of flow for glymol ranged very low, the highest being less than 30% for a head of 0.6 ft. It is clear that no satisfactory results can be obtained by using the flow of this medium

in a measuring device. Kerosene gave results very similar to those of water, but about 10% less in amount.

A difficulty of appreciable moment in connection with this device is that involved in inducing a current or jet of liquid or gas of less specific gravity than water to discharge into the latter without sacrificing any of the head due to velocity of the stream which is being measured. It is to be borne in mind that the apparatus must necessarily operate under extremely small heads, such, for instance, as 0.01 or 0.02 ft., as due to velocities as low as 1 ft. per sec. or less. Therefore, there is no energy to spare in the velocity head of the stream being measured, all of this head being required for the purpose of recording; in other words, the operation of the apparatus itself must be carried on with power entirely outside of the static or velocity head of the stream.

If a vessel containing water is superimposed on a tight vessel containing air, and if these be connected by an orifice less in diameter than about 0.011 ft., it will be found that the water will not flow down through the orifice after the air in the lower vessel has been compressed to a density sufficient to resist the pressure of the superimposed water. Surface or skin tension of the water itself, no doubt, has much to do with this resistance, but, whatever the reasons are, it will be found that the water pressure is balanced by the air pressure plus this frictional resistance. If the orifice is made larger in diameter than 0.011 ft., there will begin to be a tendency for the air and water to pass each other through the orifice, the air escaping by bubbles and marked disturbance of the water. With similar orifices, however, there is no passage of air and water, and a condition of equilibrium is maintained.

If oil is substituted for air in the lower vessel similar results will be observed, but to a much more marked degree, depending on the viscosity of the oil. The repulsion between oil and water and the surface tension of each medium creates a balance of forces, even with connecting orifices of very considerable size between the vessels, so that the heavier medium will not sink into and displace the lighter one. This refusal of the different media to pass each other through small orifices is a feature of importance in working out the measuring device herein referred to.

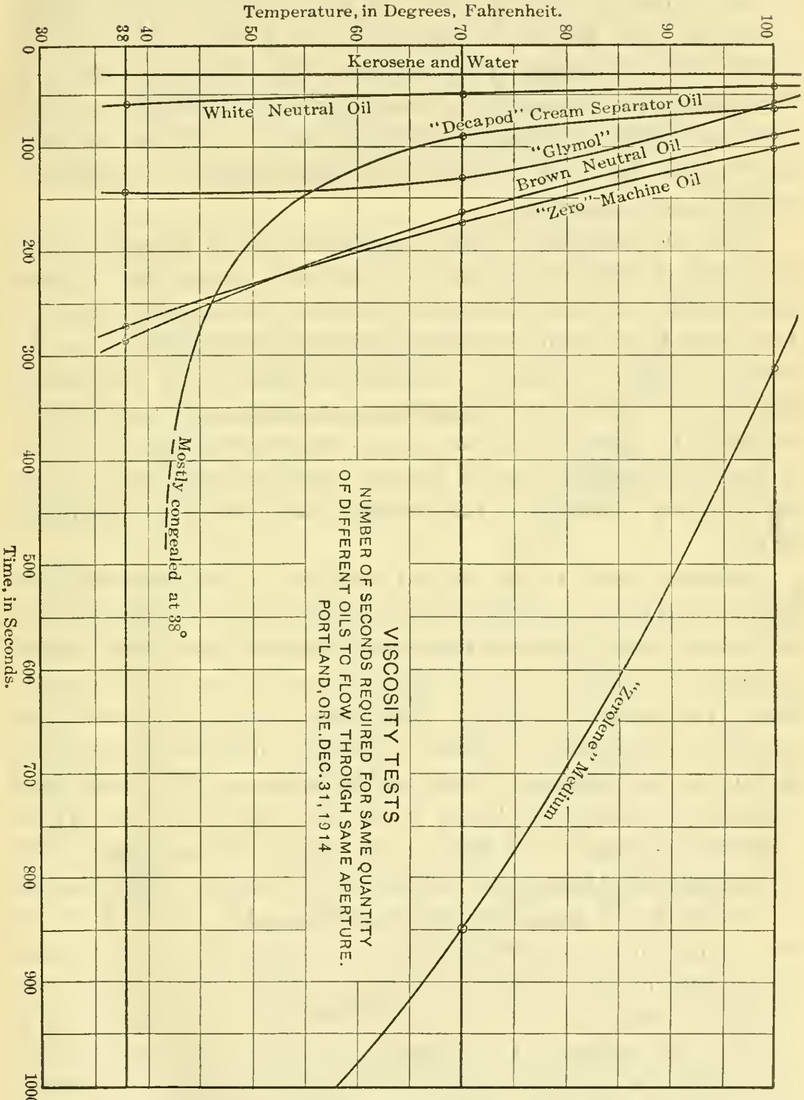

Temperature, in Degrees, Fahrenheit.
30
38
40
50
60
70
80
90
100
0
100
200
300
400
500
600
700
800
900
1000
Time, in Seconds.
Kerosene and Water
White Neutral Oil
"Decapod" Cream Separator Oil
"Glymol"
Brown Neutral Oil
"Zero"-Machine Oil
"Zerolene" Medium
Mostly congealed at 38°
VISCOSITY TESTS
NUMBER OF SECONDS REQUIRED FOR SAME QUANTITY
OF DIFFERENT OILS TO FLOW THROUGH SAME APERTURE.
PORTLAND, ORE. DEC. 31, 1914

FIG. 7.

It was found by experiment, moreover, that a very appreciable force was necessary to break the seal of surface tension and induce a flow between differing media through the small orifices required in this apparatus. The amount of this force differs with the size of the orifice and the character of the medium. With an orifice 0.001 ft. in diameter, a head of nearly 3 in. was necessary to overcome this resistance and induce air to pass upward into water. With oils the resistance is less than with air, but in all cases there is an appreciable resistance to be overcome.

From the foregoing it will be understood that a substantial operating head or force must be provided from other sources than the static and velocity heads of the flowing stream. In the volumeter, this force has been obtained from gravity operating between the two vessels, in which the heavier medium is at the top and consequently tends to sink to the bottom and displace the lighter medium there contained. The apparatus provides means whereby the force of gravity is exactly controlled so as to furnish operating power and to apply the necessary correction to the discharge coefficient of the controlling orifice.

Referring again to the reduced coefficient of the flow with these small orifices under low heads, a study of the curves of coefficients seems to indicate clearly that the reduction is largely due to some obscure retarding force, such as surface tension, which actually absorbs a portion of the operating or velocity head applied, and is, to a considerable extent, if not entirely, a constant in itself. Thus, with heads as low as 0.01 ft., the coefficient of flow may be as low as 50%, which would indicate the velocity of the jet to be about 0.4 ft. per sec. If the coefficient had been about 90%, as in the case of the higher heads, the velocity would have been about 0.7 ft. per sec. Thus the loss in velocity due to surface tension or some similar cause is about 0.3 ft. per sec., an amount which would correspond with an approximate loss of head of about 0.007 ft. Thus the addition of a constant correction of some small amount like this had been found practically to eliminate the variant in the discharge coefficient for the small orifices, and to give a resulting accuracy to the device, which is forcibly brought out by the curve of discharge shown in comparison with the curve of theoretic velocities and discharge in Fig. 8. These results are of the highest practical importance.

In the determination of discharge of the volumeter for different heads, as given in Fig. 8, the following method was used: The apparatus was set up on a table, the influent and effluent pipes being connected by ¾-in. rubber hose with two vessels open at the top and filled with water. The differences in head between the water levels of these vessels was carefully measured by a hook-gauge, and the quantity of water actually passing into and out of the apparatus was measured

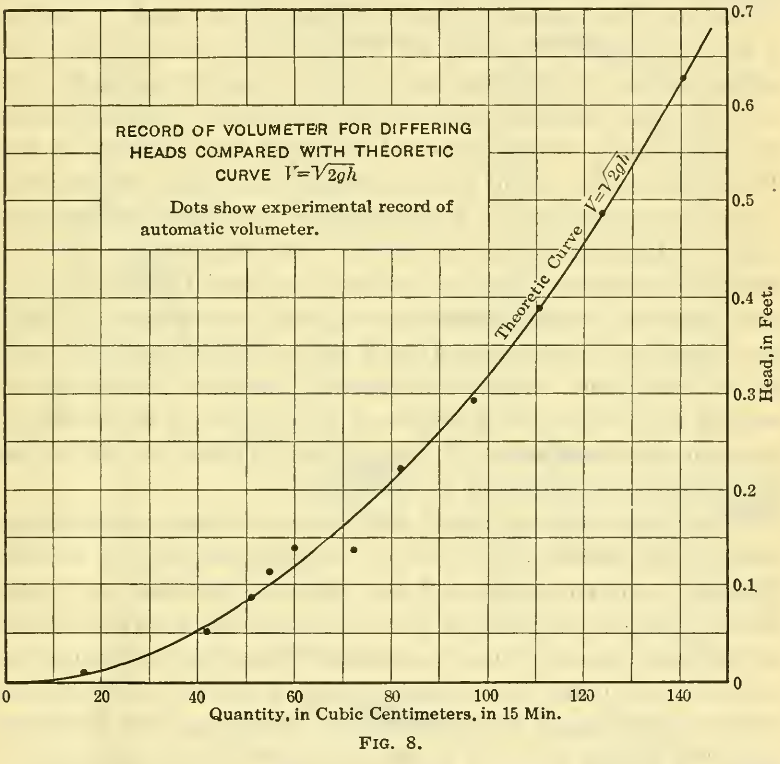


FIG. 8.

by accurate glass graduates and checked by the scale on the volumeter itself.

Extreme accuracy was possible in all measurements, and the latter were carefully checked and repeated a number of times. On Fig. 8 the quantities passing are expressed in cubic centimeters per 15 min. This is a purely arbitrary measurement, and might as well be expressed in some other form; actually, the quantities measured

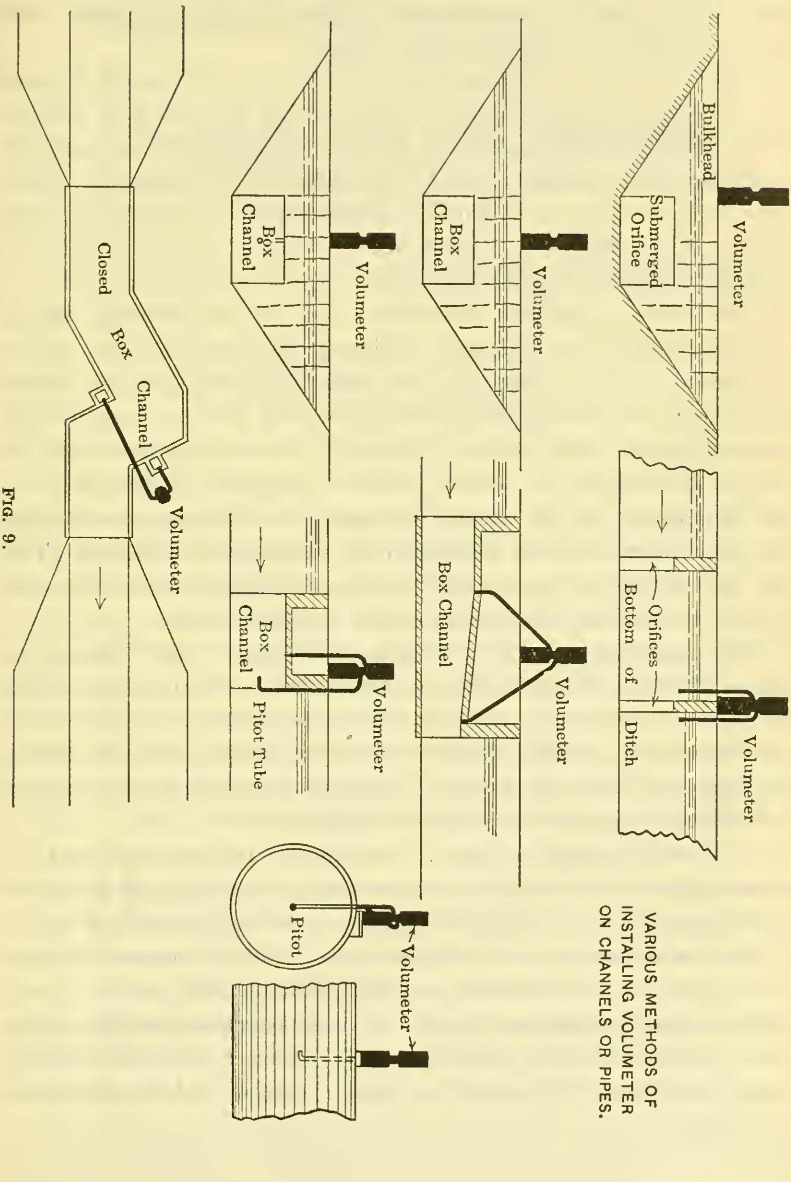
were the result of much longer runs than 15 min., usually being 1 or 2 hours in each case.

Referring again to Fig. 1, it will be seen that if C is at a higher elevation than A, then the pressure on the lower side of C, communicated from A through the light medium in the lower vessel, will be greater than that communicated to the upper side of C through the water in the upper vessel. The greater the distance C is above A, the greater the discrepancy in the balance of forces on both sides of C will be. The reverse of these conditions is also true. Therefore, it is only necessary to adjust the height of the orifice, C, above the influent orifice, A, to obtain, not only the necessary operating force, but such small additional head as may be required to overcome frictional resistances at the orifice and to correct the coefficients of flow. This adjustment is one of the most important features of the device.

In actual experience it has apparently been found advantageous to have a little air in the two vessels, to furnish operating elasticity. When the vessels are filled up solidly with liquid there seems to be some tendency toward stiffness and rigidity in operation, so that a movement once communicated tends to be unduly continued; and, on the other hand, there is an apparent inertia to be overcome in starting up. With a small cushion of air or vapor in the vessels, the apparatus can be adjusted to a very delicate balance, so that a head as low as 0.001 ft. will start it recording.

The principal sources of error in this device are temperature changes affecting the specific gravity of the measuring medium or its bulk. Changes in the air density will also influence the results to a small extent. Some of the causes of error can be avoided or obviated; others are inherent, but, to a very considerable extent, tend to balance and neutralize each other. The important point, however, is that the errors involve a very small percentage of the record and are practically negligible, as may be noted by the record of operation appearing on Fig. 8.

The practical operation of the device should probably be first for irrigation uses, and particularly for measurements of individual service flows or flows in small laterals. The method of erection and attachment which appears most simple and effective would be to use a section of box flume or pipe, a few feet in length, at the point of measurement, or two submerged orifices, as shown on Fig. 9. The



VARIOUS METHODS OF INSTALLING VOLUMETER ON CHANNELS OR PIPES.

FIG. 9.

device would be connected with the stream by permanent iron pipe connections and protected by a small box cover. Recharging of the apparatus could be accomplished without disturbance other than merely emptying one vessel into the other.

The device records volume at any given time. No record is made of the various changes in rates of flow during the period of observation. If this is desired, however, it is a mere matter of adding clock-work recording mechanism. A continuous roll of sensitized paper passed by the gauge will enable a photographic record of variations in flow to be obtained. Such a record will be a mass-curve of the flow during the period of observation.

The device, as applied to irrigation use, has the advantage that it is independent of such matters as drifting sand, weeds, or any floating or suspended trash. Nothing in suspension will pass into the influent pipe, which has practically no velocity of flow and has, moreover, an upward course. Solid matter if placed in the influent pipe would in fact settle back into the stream. Even if suspended or floating material should get into the measuring vessels, it would not pass through the controlling orifice on account of the internal arrangements. This has been put to very severe test in the experimental run, as the water used in this was full of impurities and suspended matter.

An apparatus like this, of course, must be set with skill by one who understands the principles of its operation. With proper setting, its further operation is so simple and automatic that any ditch rider or mechanic of average intelligence will be able to reset and read it each day or whenever necessary. The device is as near a self-contained and self-operating one as can well be imagined.

The device should be cheap. It ought to be made and sold for a few dollars, which should cover everything, including all the necessary pipe and fittings. This should place it within the reach of all.

For use on water-works systems or for industrial purposes its simplicity and cheapness should render it valuable and enable a more intelligent understanding to be had of many operating conditions that have hitherto not been fully understood owing to the cost of large meters and the skill required in their operation and the deduction of results.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE CHERRY STREET BRIDGE, TOLEDO, OHIO

By Clement E. Chase, Jun. Am. Soc. C. E.

To be Presented December 1st, 1915.

Synopsis.

This paper describes the general design and the most interesting and instructive features of the construction of a massive reinforced concrete arch bridge across the Maumee River at Toledo, Ohio.

After a short résumé of the history of the former bridge at this site and of the new bridge project, the loadings and general features of the design are stated. The construction work is followed in some detail from its beginning in 1910 to the opening for use of the first half of the structure during the winter of 1912-13. The construction of the second half of the bridge, being to all intents a duplication of the work on the first half, is merely outlined up to January 1st, 1914, at which time the structure was practically complete.

The description of construction deals with the following features: The building of the bridge in parallel halves, carrying traffic on the old structure at the same time, until the first half was completed; the use of deep, single-wall coffer-dams and attempts to dig wells to rock from the bottom of the pumped-out dams by the Chicago method; change of plans by reason of the proven hazard of this first plan and the utilization of steel cylinders sunk inside the unwatered dams; the

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings,* and, when finally closed, the papers, with discussion in full, will be published in *Transactions.*

filling of these cylinders with concrete and the sealing of the coffer-dams by concrete deposited under water; the construction plant and methods of depositing concrete under water, with a statement of the general principles; the collapse of a steel coffer-dam, through failure of the bracing, and its repair and re-use; the use of movable steel centers to support the arch concrete, and the expedients resorted to for increasing the width (of the arch barrel) poured at one time; the forms, plant, and methods of work in building the arches, spandrel walls, and viaduct; the procedure followed in water-proofing and back-filling; and the erection of the bascule span. Special attention is paid to the esthetic appearance of the bridge. The paper closes with a summary of unit prices, quantities, and amounts expended for the different parts of the work.

Historical.

The main portion of the City of Toledo, Ohio, is separated from an important industrial and residential section, known as East Toledo, by the Maumee River, across which, from the earliest history of the community, the Cherry Street Bridge has been the main thoroughfare. Following several predecessors—of types corresponding to the progress of the art of bridge building in the United States—the iron structure, which has been recently replaced, served to carry the growing traffic across the river from 1883 until January, 1913. Several years before that time, it was realized that the bridge was inadequate for the constantly increasing loads which it had to carry, and that, with the larger number of lake carriers docking above its site, a more rapid form of movable span was needed than the existing swing span. The question of type and location for the new bridge was debated heatedly, at great length, and with such intense partisanship that the whole project was held up for a long period. The main points at issue were finally settled—the new structure should be of concrete, with a double-leaf lift-span across the channel, and should cross the river, as before, at Cherry Street.

Design.

As finally located, the south half of the new and much wider structure covers the site of the old one. For a time it was planned to construct temporary piers alongside the old ones and shift the

spans sideways so as to clear the bridge site. Later, this plan was abandoned, largely on account of anticipated danger from ice, and it was decided to build the new bridge in longitudinal halves. The north portion was to be completed first, the old bridge was then to be removed, and finally the arches were to be extended to their full width of 73 ft. 10 in. The width of Cherry Street is maintained practically unchanged as it passes over the bridge, the roadway being 52 ft. between curbs and the sidewalks 9 ft. wide. Two car tracks are provided in the center of the roadway.

There is a clear channel opening of 200 ft., closed by a double-leaf bascule span. Approaching this, the roadway has a grade of 2.02% from the east and 1.35% from the west, which gives enough elevation at the bascule to allow a clearance of 29 ft. under the steel, with span closed. This is quite enough to pass the tugs and "sandsuckers" which caused a considerable percentage of the openings of the old draw-span. Maintaining river traffic through both bridges during the construction of the new one, required the layout of the new channel as shown on Fig. 1, so that in order to open it, the old draw might be swung one way until the new bridge should go into commission. Fig. 1 also shows the layout and principal dimensions of the arch openings. To the east of the channel there are five spans, decreasing in length from 108 ft. to 66 ft. 6 in. and in rise from 28 ft. 6 in. to 16 ft., in order to conform to the grade. To the west there are two more of the 108-ft. arches—the grade here being taken care of by dropping both ends of the arch nearest the shore by 1½ ft. These 108-ft. arches have a thickness of 7 ft. at the springing line and 2 ft. 3 in. at the crown. Their intrados is an ellipse with a semi-major axis of 54 ft. and a semi-minor axis of 29 ft. Fig. 2 gives the dimensions of the various arch rings. The assumptions of load used in the design include a sand fill weighing 100 lb. per cu. ft.; 50-ton interurban cars in trains on both tracks, each covering a width of 10 ft.; and a live load of 150 lb. per sq. ft. on the remainder of the roadway and of 100 lb. per sq. ft. on the sidewalks. A maximum stress in the concrete of 450 lb. per sq. in. was specified, and a temperature range of 70° was assumed. The reinforcement consists of latticed ribs, 2 ft. 6 in. from center to center, and transverse rods, along the extrados and intrados, 3 ft. apart.

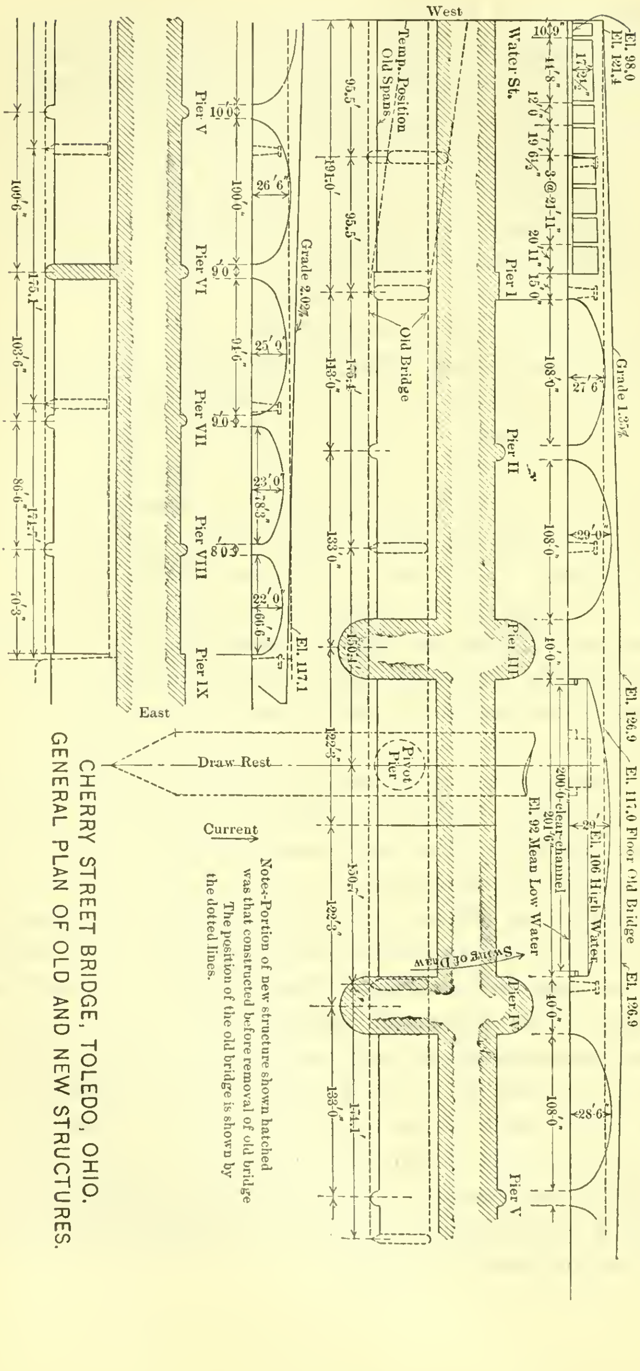


FIG. 1.

CHERRY STREET BRIDGE, TOLEDO, OHIO.
GENERAL PLAN OF OLD AND NEW STRUCTURES.

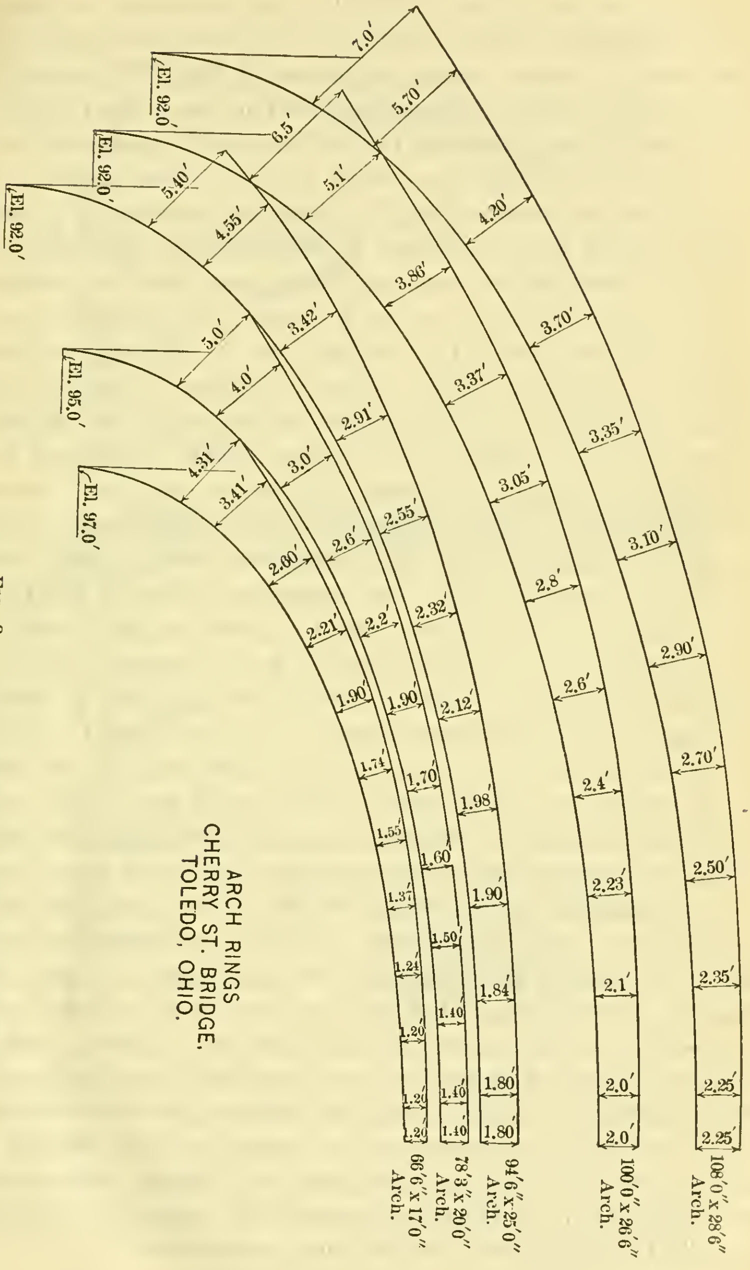
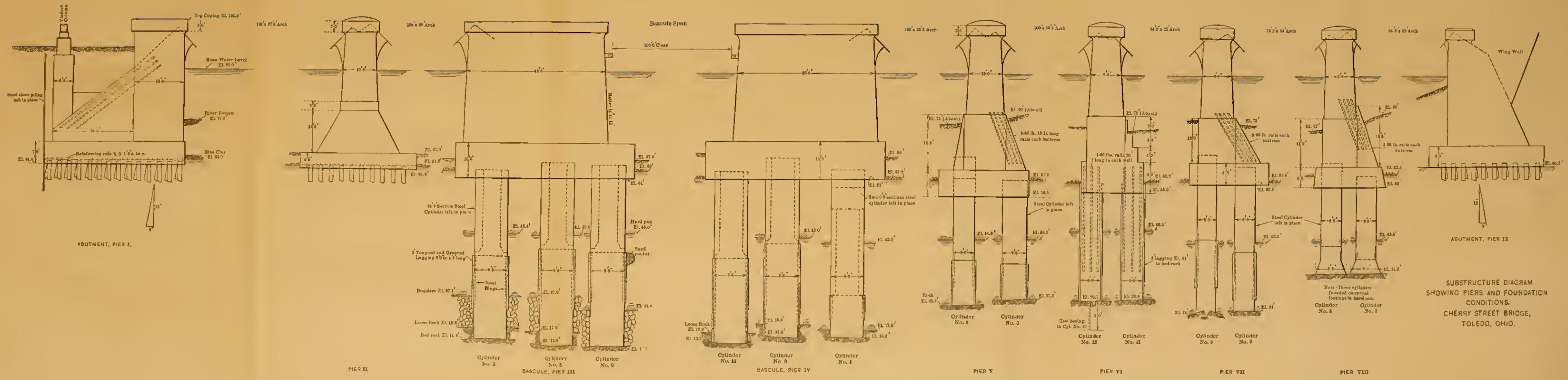

ARCH RINGS
CHERRY ST. BRIDGE,
TOLEDO, OHIO.
El. 92.0'
El. 92.0'
El. 92.0'
El. 95.0'
El. 97.0'
66'6" x 17'0" Arch.
78'3" x 20'0" Arch.
94'6" x 25'0" Arch.
100'0" x 26'6" Arch.
108'0" x 28'6" Arch.

Fig. 2.

Of the nine piers (Plate XXXVII), three are carried on piling driven to firm bearing in the hardpan and the remaining six on cylinders which extend to bed-rock, except in the case of Pier VIII, where the cylinders were stopped at hardpan and belled out for a larger footing. Rock is found at from Elevation 13.7 to Elevation 27.3, measured from the datum, 92.0 ft. below mean water level, and rises toward each shore from the low point at Pier IV. Above the rock there is a layer of hard, gravelly clay or hardpan to about Elevation 44. Between this and the sand which forms the river bottom, there is a stratum of soft, plastic, blue clay, about 20 ft. thick. The pier shafts were carried below the river bed to the blue clay in all cases, and are supported at that point either by piles or cylinders. For the piers between the spans of unequal length, on the east side of the channel, buttresses, reinforced with rails, are placed on the side toward the smaller arch, covering the cylinders outside the pier shaft. Below mean water level a $1:3\frac{1}{2}:7$ concrete was used, with embedded stone, the quantity being limited by the specifications to 33%, though less than this was actually placed. The smaller piers (Plate XXXVIII), which vary from 8 to 10 ft. in width, are carried on from eleven to twelve cylinders, some 6 ft. and some 7 ft. in diameter. The two large bascule piers, 40 by 126 ft. over all, are supported on eleven cylinders, 9 ft. 6 in. in diameter. The two abutment piers, I and IX, are carried on piles, 3 ft. from center to center and 25% of these piles are driven to a 2 in 12 batter. Pier II, which has a 108-ft. arch on each side, is carried on 370 piles, supporting a 24 by 105-ft. footing.

The construction of the bridge in longitudinal halves necessitated a central spandrel wall to retain the fill in the north half until the completion of the whole bridge. For this, a reinforced concrete wall, 18 in. thick, with counterforts 9 ft. from center to center, was designed. The outside spandrel wall was shown on the original plans —on which the contractor's bid was based—as a gravity section. and was built with a base forty-two one-hundredths of the height and a fixed top width of 1 ft. 6 in. The back surface of the wall is warped, as the batter changes constantly, but this caused very little difficulty in forming. A tile conduit to carry power and lighting cables across the bridge rests on a reinforced concrete slab supported under the sidewalks by brackets built into the outer spandrel wall.

PLATE XXXVII
PAPERS, AM. SOC. C. E.
OCTOBER, 1915
CHASE ON
CHERRY STREET BRIDGE, TOLEDO, OHIO



SUBSTRUCTURE DIAGRAM
SHOWING PIERS AND FOUNDATION
CONDITIONS.
CHERRY STREET BRIDGE,
TOLEDO, OHIO.

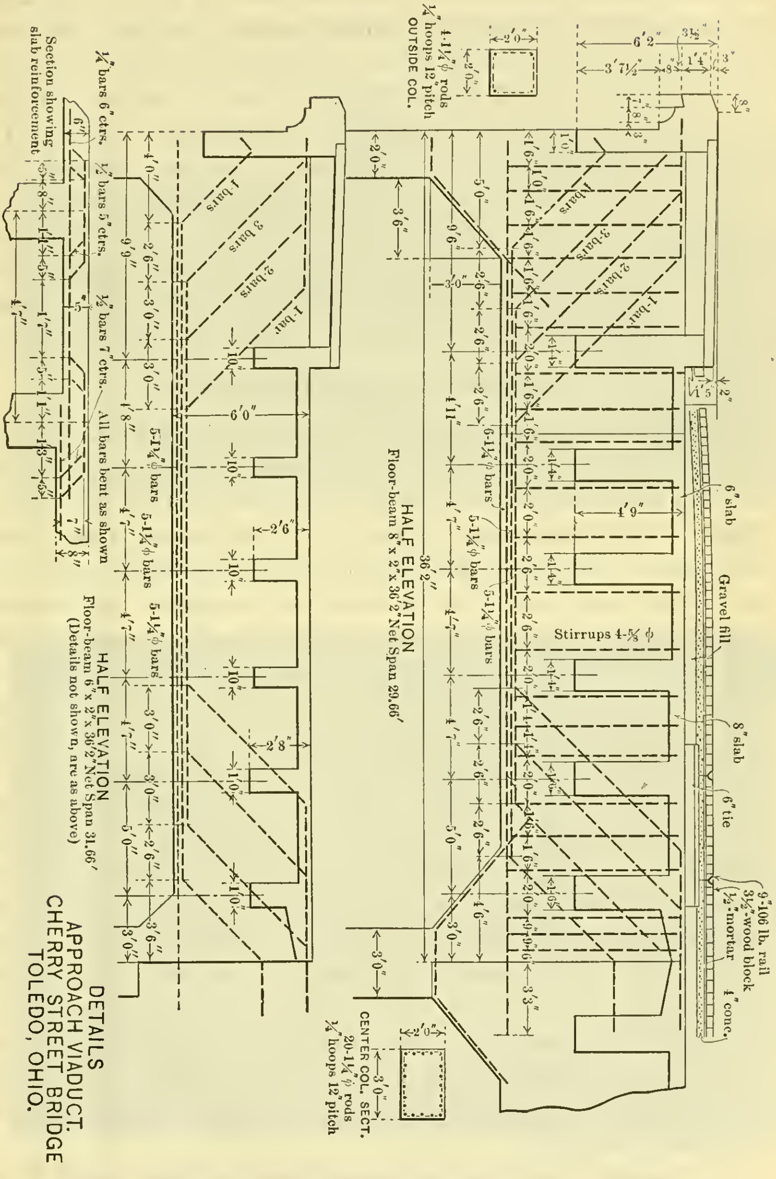

DETAILS
APPROACH VIADUCT.
CHERRY STREET BRIDGE
TOLEDO, OHIO.
HALF ELEVATION
Floor-beam 8" x 2" x 36' 2" Net Span 29.66'
HALF ELEVATION
Floor-beam 6" x 2" x 36' 2" Net Span 31.66'
(Details not shown, are as above)
4-1¼" φ rods
¼" hoops 12" pitch
OUTSIDE COL.
CENTER COL. SECT.
20-1¼" φ rods
¼" hoops 12" pitch
Section showing
slab reinforcement
¼" bars 6" ctrs.
½" bars 5" ctrs.
½" bars 7" ctrs.
All bars bent as shown
Stirrups 4-⅝ φ
6" slab
Gravel fill
8" slab
6" tie
9"-106 lb. rail
3½"-wood block
½"-mortar
4" conc.

FIG. 3.

From the abutment on Water Street, the roadway is carried over the street and the Manufacturers Railroad tracks to Pier I, just inside the harbor line, by a heavy reinforced concrete viaduct. The span over Water Street is 44 ft. 8 in. in length, the girders under the street-car tracks being T-beams, 18 in. wide and 57 in. deep, with an 8-in. slab. Except for two short 10 and 12-ft. spans adjoining the long one, the panels are 21 ft. 11 in. long. Each bent contains three columns, 35 ft. 2 in. from center to center. Very massive floor-beams, shown by Fig. 3, support the stringers over Water Street, the dimensions being 2 by 8 ft., and the reinforcement is correspondingly heavy. The other floor-beams have a 2 by 6-ft. section. The columns have footings of concrete supported on oak piles cut off below ground-water level.

Water-Proofing and Expansion.

Special precautions were taken to prevent discoloration and disintegration of the walls and arches by seepage. In the valley, between each pair of arches, four 6-in. cast-iron pipes serve to drain the spandrel filling. There are scuppers in the curb at the crown of each arch. The backs of the arches and the inside of the spandrel wall up to the sidewalk are covered with a water-proof membrane consisting of five-ply Barrett felt and six coats of Barrett specification pitch. (See Fig. 4.) At all sharp angles and at the longitudinal construction joints in the arch ring, this membrane is reinforced with two layers of canvas. Furthermore, the water-proofing is protected by a layer of brick placed flat over the arch barrels, with open joints, except in the steepest part of the slope. A wall, one brick in width, was also built against the face of the spandrel walls, where they were water-proofed. At each side of each pier, and masked behind the pier face, there are expansion joints in the spandrel wall. These joints were painted with pitch on the sliding surfaces and filled with felt between abutting surfaces, to form a compressible layer. A fold, reinforced with canvas, was provided in the water-proof membrane at each joint. The wall was so low at the crown of the arch that although an expansion joint was provided there, it was hardly necessary. Where the hand-rail was cut, soft sheet-asbestos filled the opening less conspicuously than felt.

The spandrel walls were reinforced throughout their middle-thirds, in order to render the formation of cracks at that point less likely.

Fig. 4.—Placing Water-Proofing Over Back of Arches and Inside of Spandrel Wall.

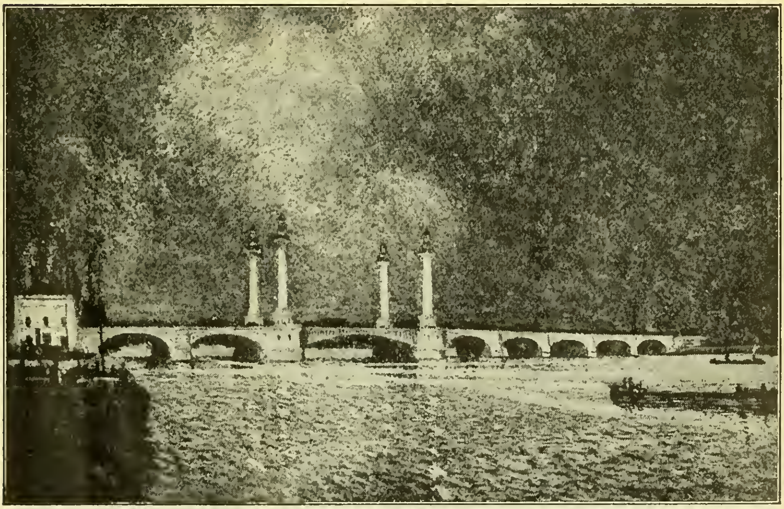


Fig. 5.—Architect's Drawing of Completed Cherry Street Bridge.

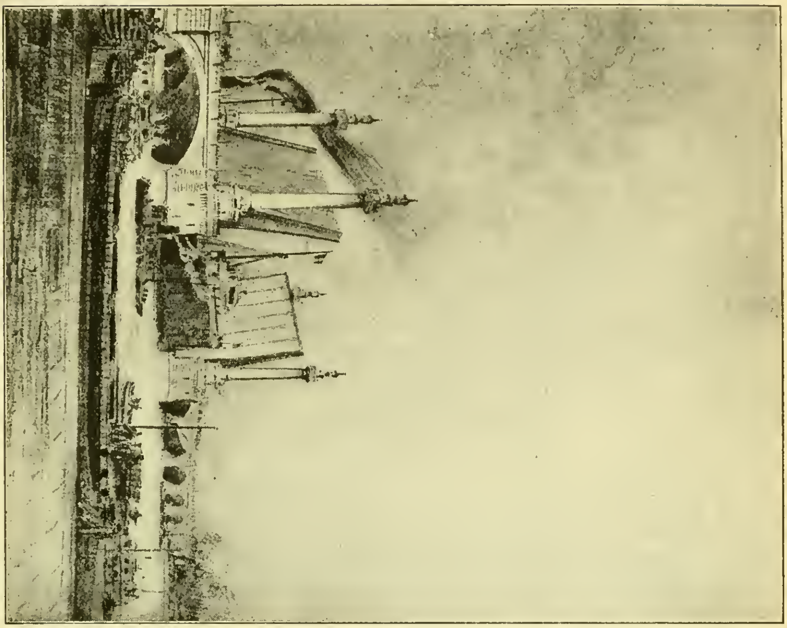


FIG. 6.—ARCHITECT'S DRAWING: FREIGHTER PASSING THROUGH OPEN BASCULE.

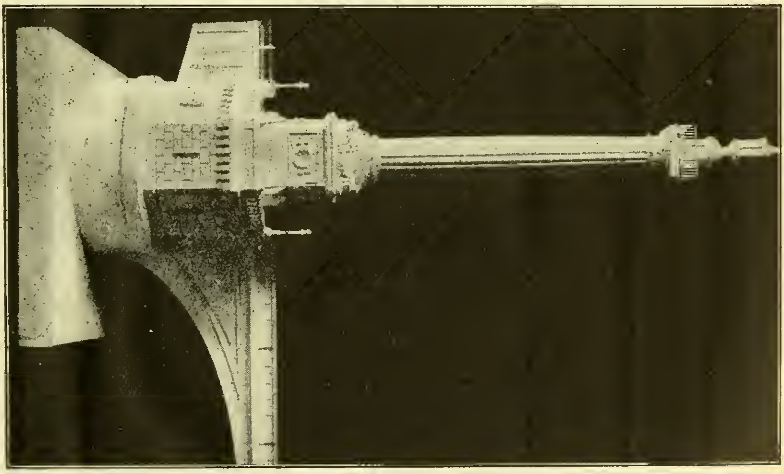


FIG. 7.—ARCHITECT'S MODEL OF BASCULE PIER AND COLUMN.

In one case, this was unsuccessful, as one of the joints at the main bascule pier (Pier IV), with an unusually large sliding surface, failed to work, and a crack resulted in the spandrel wall at about the quarter-point.

ARCHITECTURAL TREATMENT.

Largely due to the far-sightedness of the Hon. Brand Whitlock, then Mayor of Toledo, in securing the appointment of a consulting architect of broad experience in public work to co-operate in the beautification of the design, the Cherry Street Bridge when completed on this plan will be one of the notable structures in the United States. With the strong and graceful outlines of the engineer's design, and with little in the way of applied ornament, the treatment brings out the natural beauties of the structure by a judicious use of the shade effects of paneling and moulding. The attention of the observer is naturally directed to the lift-span, which, when closed, forms an arch almost twice the length of the adjacent ones, and when open, is the most striking object in the harbor. Realizing this, it is made the central feature of the design, and four tall columns, or towers (Fig. 5), rising from the two massive bascule piers, direct attention to it. Without these the open position, with erect leaves breaking the sweep of the arches, gives an appearance of awkwardness—a utilitarian distortion of the structure. By the use of the towers, which dominate the leaves, and the massive piers, the effect of a gateway to the city is secured, and the passing of one of the mammoth lake freighters through the bridge (Fig. 6) will be the climax in its beauty.

The details of the tower and bascule pier design are shown by the architect's model, Fig. 7. The towers (unless the present City Administration sees fit to change the plans), will rise 150 ft. above the water and be topped by powerful lamps. The bases of two serve as operators' rooms and those of the others as public comfort stations. The handrail is very massive, not pierced except over the bascule piers, and is broken by panels and offsets at the piers and at regular intervals between. The pier noses are half round, up to Elevation 106, above which point, the pier is flat, set out from the wall 1 ft. 4 in., and marked with horizontal rustications. (See Fig. 8.)

The surface treatment of the bridge consists of roughening certain portions, exposing the special aggregate of small gray and yellow limestone chips which was cast against the forms, and, for contrast,

leaving untreated or smooth-rubbed, the bands and mouldings. The lighting system first contemplated consisted of a line of metal columns at close intervals along each hand-rail. Each column was to carry a single large globe containing a cluster of lights. Further consideration of the probable cost of lighting the structure under this plan forced a change. The system adopted has somewhat similar columns, but, with less powerful lights, and at longer intervals, along the curb.

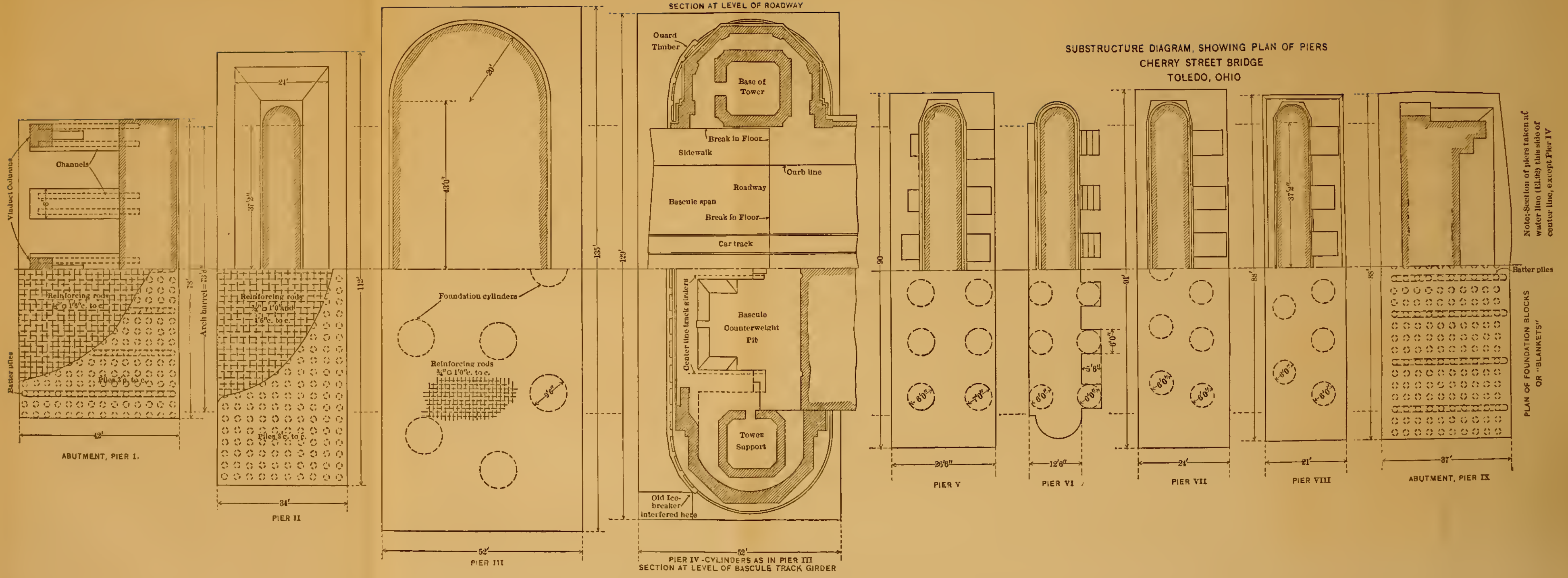
Construction.

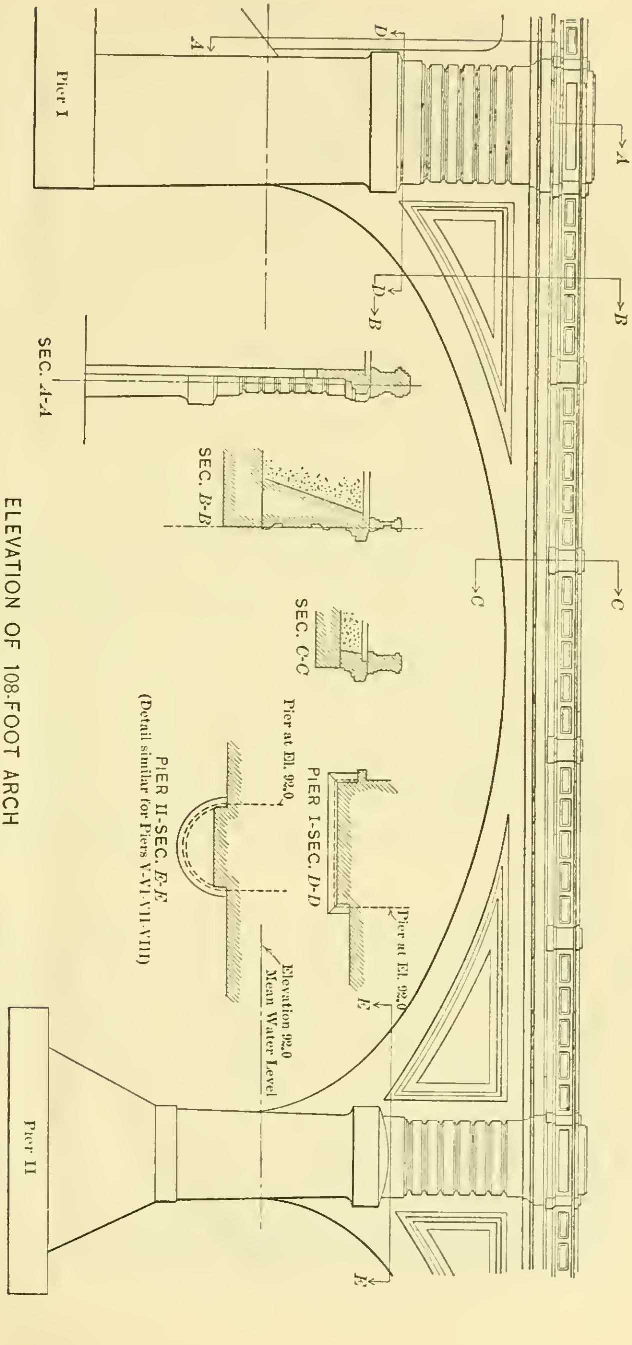
As might have been expected, the construction was very much complicated and lengthened by the necessity of maintaining traffic on the old bridge during the building of the new one. Although the superstructure had to be constructed in halves, it was first planned to build all the piers their full length and, where necessary, to carry the old bridge on temporary trusses and girders across the coffer-dams. According to the original plan for foundations, provided in the design, cylinders were to be put down to rock by the "Chicago" method, by first pumping out the coffer-dams and then digging the wells in the open, lagging them with tongued and grooved staves held in place by iron bands, as the excavation proceeded.

This daring and unusual procedure in work of this nature would have involved unwatering single-wall coffer-dams up to 53 by 152 ft. in size against a head of 30 ft. of water, and then starting from the bottom thus exposed and digging from 12 to 24 wells as much as 45 ft. farther, to rock. The work would have been through a blue clay, shown by the borings to contain pockets and streaks of sand.

Work was begun under this plan on Pier VI, on August 9th, 1910. A single-wall coffer-dam, 24 by 90 ft., of steel sheet-piling 46 ft. in length, was driven around the pier site. For that part of the dam under the old bridge, the sheeting was assembled complete for each side and floated in, suspended in a vertical position from staging built on a barge. To drive it, a heavy forging was hung from the top of the pile, which allowed the hammer to work several feet below the pile top. The timbering was placed as the dam was unwatered, and was very heavy, consisting of seven racks, 4 ft. 3 in. apart vertically. The contractor's full pumping equipment not having arrived, there was considerable difficulty in getting the water "started down" in this dam. Three sand dredges, equipped with two 8-in. and one 10-in.

SUBSTRUCTURE DIAGRAM, SHOWING PLAN OF PIERS
CHERRY STREET BRIDGE
TOLEDO, OHIO





ELEVATION OF 108-FOOT ARCH
PIER I TO PIER II
FIG. 8.

centrifugal pumps, were finally pressed into service to aid the 8-in., steam-driven centrifugal and two reciprocating pumps of the contractor's plant. As soon as a difference of head was secured, a mixture of manure and ashes, dropped along the dam on the outside, successfully stopped the leaks. Grouting through holes in the sheeting was also resorted to where the leaks occurred below the level of the river bed on the outside. After the excavation and timbering were completed, the twelve wells were started in the bottom of the coffer-dam and successfully sunk to rock, the work being done in a constant shower of water in many cases and despite the hazards of the method, which were fully realized by the contractor's men. Continual pumping was required in several of the holes, and emphasized the danger of a blow-out. At one time, it seemed as though it would be necessary to excavate a couple of the wells through water, have a diver clean off the rock, and then deposit concrete under water. The leakage was finally controlled by grouting through pipes driven down alongside the cylinders on the outside and through holes drilled through the lagging. As soon as they were completed, the wells were concreted to a point 1 ft. above the bottom of the pier excavation. Six 60-lb. rails were placed in the upper portion of the cylinder as reinforcement. The pier shaft was carried up in successive lifts to the under side of each rack of timbering. After waiting 48 hours in each case, the dam was braced against the concrete, the rack removed, and the concreting continued. In subsequent dams, where much less timbering was used, the struts were concreted into the pier to within 10 ft. of normal water level, Elevation 92, and were afterward cut off flush with the face of the pier.

At about the same time, work was begun on Piers IV and IX, at both of which temporary supports had to be provided for the old spans, to allow the piers to be removed. At Pier IX two plate-girder spans, and at Pier IV four timber Howe trusses, were utilized to support the spans over the center of the coffer-dams. Great difficulty was experienced in driving the dam at Pier IX, the steel piling being deflected, buckled, and opened by the obstacles encountered. Work on this pier, though begun at this early date, was not finished until May 9th, 1912. Ultimately, an outer wall of Wakefield sheeting had to be driven around the worst portion of the dam. Under the bridge, where it was necessary to drive the sheeting down through the floor, one pile at a time,

trouble was caused by the sheeting leaning. Special wedge sections were made and driven, in an effort to keep the piling vertical. In the other dams, where all the sheeting was set and driven successively a few feet at a time, this trouble was prevented.

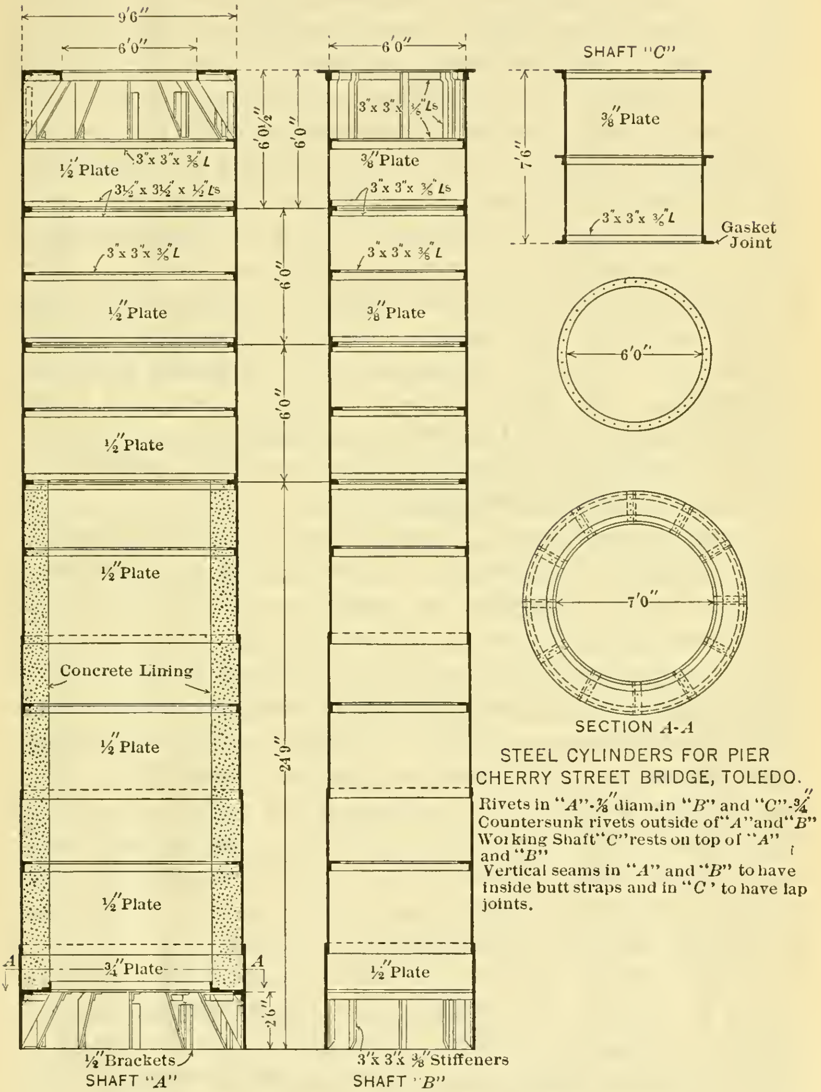
The large size of the bascule pier (Pier IV) necessitated a coffer-dam 53 by 132 ft., which was started in August, 1910, with the intention at first of completing it before the break-up of the winter of 1910-11. A bulkhead was driven across the dam, in order to allow the south half of the pier to be rushed to completion and furnish support for the old spans, but the work had not progressed very far before the ice went out of the river, without doing any damage to the temporary support of the spans. The bulkhead was therefore pulled and the timbering of the entire dam started. Three racks were placed before unwatering, each rack being assembled in turn above water and jacked down to position. When the coffer-dam was pumped out, it soon became apparent that insufficient provision had been made to secure the long struts against sagging, with the result that a couple of the overstressed timbers failed. A fourth rack was hurriedly placed 2 ft. below the third, and the timbering was shored and trussed up. As excavation proceeded, two more racks of timbering were placed and, on June 8th, 1911, the desired elevation of bottom (61.0) was reached and excavation started in three wells. On the afternoon of June 9th, water was discovered coming up through the bottom of the dam at a point about 10 ft. from the coffer-dam, increasing in quantity until pumps with a combined capacity of about 4 000 gal. per min. had been started to control the flow. Realizing then the gravity of the situation and the imminent danger of a blow-out, the contractor shut down all the pumps and allowed the dam to flood.

A consultation was then held between the contractor and the officials and consulting engineer of the City, at which the verdict was reached that it would be unsafe to apply to the large coffer-dams of Piers III and IV the methods first planned, and used already with great difficulty and danger in the construction of Pier VI. For the big piers, it was decided to use eleven steel cylinders, 9 ft. 6 in. in diameter, in place of the twenty-four lagged wells of 6 and 7 ft. diameter. These cylinders were to be sunk inside the coffer-dams before the latter were to be filled with concrete up to the elevation of the base of the pier, and in case water was encountered in quantities which made it necessary.

Below the elevation at which the cylinders were to be stopped the wells were to be carried down to rock with timber lagging, as in the original plan. After rock had been reached, the cylinders were to be filled with concrete up to the elevation of the base of the pier, and then the section of steel cylinder from there to the water's surface was to be removed. A layer or "blanket" of concrete, 10 ft. thick, was then to be placed over the entire area of the coffer-dam, being deposited under water by suitable means. The dam would then be timbered, pumped, and the pier shaft completed. Piers V, VII, and VIII were to be founded according to the original design, but, in order to expedite the work and prevent accidents, steel cylinders were substituted for lagging. To offset the increased cost of the foundations, piles were substituted for cylinders under Piers I, II, and IX, Pier II having little unbalanced thrust and the abutment piers affording an opportunity for a large spread of footing and massive piers.

The steel cylinders for Pier IV, as called for under these plans (Fig. 9), had a cutting-edge section 29 ft. 6 in. in length, to which could be bolted additional sections 6 ft. in length. A shelf around the inside of the bottom section near the cutting-edge supported the lining of concrete, 1 ft. 3 in. in thickness, which furnished part of the weight required for sinking. A wooden bulkhead was placed in the cylinders to float them while the lining was being placed, and was removed after the weight of the concrete had caused the cutting-edge to seal itself in the clay. As the cylinder followed the excavation down, additional top sections 6 ft. in length were added as needed. The third or fourth 6-ft. section added was a reducing section, decreasing the diameter from 9 ft. 6 in. to 6 ft., and allowed the placing of concrete weight blocks around the 6-ft. cylinder to get the needed sinking weight. The blue clay was cut with spades and removed in small tilting buckets. One stiff-leg derrick, mounted alongside the coffer-dam on piles, served all the holes except those under the bridge. The spoil from these latter was hoisted by runner lines working through snatch-blocks hung from the Howe trusses.

At Elevation 43.5 the material changed to a very hard dry clay or hardpan, which it was necessary to break up with a mattox or split off with bars and wedges. Limestone boulders, of all sizes up to ½ cu. yd., were encountered in this stratum, with occasional sand pockets, only a few of which showed any traces of water. The holes in most



9'6"
6'0"
6'0"
SHAFT "C"
3" x 3" x ½" Ls
⅜" Plate
½" Plate
3" x 3" x ⅜" L
3½" x 3½" x ½" Ls
⅜" Plate
3" x 3" x ⅜" Ls
6'0½"
6'0"
7'6"
3" x 3" x ⅜" L
Gasket Joint
3" x 3" x ⅜" L
3" x 3" x ⅜" L
½" Plate
⅜" Plate
6'0"
6'0"
6'0"
½" Plate
½" Plate
7'0"
Concrete Lining
SECTION A-A
½" Plate
STEEL CYLINDERS FOR PIER
CHERRY STREET BRIDGE, TOLEDO.
24'9"
Rivets in "A"-⅞" diam. in "B" and "C"-¾"
Countersunk rivets outside of "A" and "B"
Working Shaft "C" rests on top of "A" and "B"
Vertical seams in "A" and "B" to have inside butt straps and in "C" to have lap joints.
½" Plate
A
¾" Plate
A
½" Plate
2'6"
½" Brackets
SHAFT "A"
3" x 3" x ⅜" Stiffeners
SHAFT "B"

FIG. 9.

cases were dry, trouble being usually caused by water following down alongside the cylinder after a sudden drop. At first, the steel cylinders were sunk to their limit before continuing them down to rock with wooden lagging. Later, after a Court decision that only the 24 ft. 9-in. section should be paid for, weight was taken off the cylinders as soon as all but a foot or so of this section was embedded in the clay. In continuing the wells below the cutting-edge, the shaft was excavated for distances of from 2 to 5 ft., and the hole was then lagged with short sections of double-grooved yellow pine sheeting, with radial edges and oak tongues. Steel hoops, 9 ft. 6 in. in diameter, made in three sections which bolted together, braced the lagging in place. In the last few feet to the bottom, where the material was favorable, the walls were often left unlagged. Bed-rock was found at elevations varying from 13.7 to 18.2 at this pier, and was usually a polished, flat limestone. The eleven cylinders of Pier IV were sunk a total distance of 490 ft. from July 28th to November 15th, 1911, with an average of 23½ days to each cylinder. The average daily progress, when work was not unduly delayed, was 2.7 ft.

During the next season, in putting down the wells for Pier III, several changes in the method of prosecuting the work resulted in shortening the time materially. An average interval of 17½ days elapsed on this side of the channel for the excavation of each cylinder, the work going forward at the rate of 3.1 ft. for each of the 15½ actual working days. In the first place, the sheeting was driven on only three sides of the dam, so that the cylinders could be concreted until nearly aground and then floated into position, instead of being lifted over the dam as before. The thickness of the shell of concrete was increased to 2 ft., leaving a shaft of 5 ft. 5 in., and making less additional sinking weight necessary in the shape of concrete blocks. Two derricks were provided for hoisting the spoil. Gangs of three men in each hole worked in three shifts of 8 hours each.

Concrete for the cylinders was mixed on the floating plant and chuted to a hopper, from which it was dropped from 35 to 78 ft. through a 10-in. pipe to place. Observation of the concrete in the cylinders, during stoppages in the work, showed practically no separation of materials and a very marked packing and tamping of the concrete. Water at times accumulated in leaky holes, or from over-wet batches, to such an extent that drying the mix did not take it up, and

bailing had to be resorted to. After concreting to the proper elevation in the shafts, the top sections of the cylinder were lifted off and re-used.

When the coffer-dams for Piers V and VII were laid out, it became apparent that the grillages of the old piers would interfere with the construction of the new ones for their entire length, and the contractor accordingly proposed to construct the remaining piers of the bridge, except the bascule pier (Pier III), in halves, to be suitably bonded together. This was to be done without change in contract price.

Provision was made, by grooves and projecting reinforcing rods, for bonding the second half of each pier to the first. One problem which had to be met in building the piers in halves, was that of continuing the coffer-dam after the old pier had been removed and construction of the second half of the bridge had been started. At one pier, a heavy timber bulkhead was placed from the side-wall of the dam to a groove in the side of the pier. In other cases, half a section of steel sheet-piling was concreted into the side of the pier, with provision for its removal later. In the abutment piers, a concrete bulkhead wall was built across the coffer-dam. These bulkheads permitted the pulling and re-use of most of the sheeting of the first half of the dam. The up-stream wall of the old dam was left in place to protect the bulkhead until the dam was to be extended to enclose the second half, when it was pulled, and the side-walls were continued.

Pier VII was the first in which the bottom of the coffer-dam was covered with a "blanket" of concrete before unwatering—the procedure intended at first only for the two large piers. Though Pier VIII had been successfully unwatered without this, it was thought best, as the additional cost was moderate, to place a layer of concrete under water in the remaining piers, rather than risk pumping out the dams without it, against a head of from 30 to 32 ft. For the small dams, the dimensions of which were about 25 by 48 ft., a layer 6 ft. thick was placed. This was of 1 : 3 : 5 concrete with two extra bags of cement per cubic yard of concrete in the lower part, over the tops of the cylinders, and one extra bag per cubic yard in finishing the blanket, to make up for possible loss of cement. Before starting to concrete, a diver cleaned off the tops of all the cylinder shafts, in order to insure full bearing.

The preferred method of depositing concrete under water was by tremie, chuting the concrete directly from the spout of the floating

mixer barge into the hopper of the tremie, which was handled by either the fixed or floating derricks.

Tremie work, though practiced for years, is far from being standardized, and as it is on the details of the construction and the handling of the tube that success depends, the methods used at Cherry Street may be of interest.

The tremie pipe consisted of flanged sections of 12-in., spiral, riveted pipe, which were bolted together to make up the various lengths required. The top section was short and was riveted to a pyramidal hopper of $\frac{1}{4}$-in., sheet steel, stiffened with angles and having a capacity of about 1 cu. yd. The edges of this hopper rested on a timber frame, suspended from the derrick by slings at the four corners. The discharge pipe from the floating mixer was self-supporting, and was simply held in place over the hopper by lines, in such a way that its weight did not have to be lifted when the tube was raised. A signalman, usually accompanied by the inspector, "rode" the hopper, and gave directions to the hoisting engineman.

Three things which must be most guarded against in tremie work, under conditions such as those at Toledo, are, first, the "plugging" of the pipe; second, the loss of the charge; and third, allowing the concrete to drop through water as it leaves the pipe. The first causes serious delay, disturbance of the concrete already placed, and danger to derrick and tremie from efforts to start the concrete flowing, and, usually, in the end, loss of the charge, thus making it necessary to refill the tremie. It can be avoided by using a quite wet mix, with small aggregate where possible, and by having the pipe hoisted enough to start the charge moving just as the hopper starts to fill, instead of after it is full. There does not seem to be any advantage in having a pipe larger than that used on this work, in fact, Mr. E. B. Van De Greyn* has recently stated that an 8-in. tremie was used successfully on the foundations for the City Waterway and Puyallup River Bridges, at Tacoma, Wash.

The second trouble causes serious washing of the last of the concrete in the pipe, for, as the column of concrete nears the bottom, the water rushes up through it to fill the tube. Also, refilling the tremie always causes more or less washing out of the cement. Loss of charge can only be avoided by good team work between the signalman and

* *Engineering Record,* August 15th, 1914, p. 188.

derrickman. The lowering of the pipe to stop the flow must be started in time, so that the pipe will be completely sealed, at least when the concrete has lowered in the tube to about the water level. The pipe should be lowered gently so as not to disturb the concrete already deposited. The regulation of this speed and the proper moment for giving the signals are matters requiring considerable practice on the part of the men. Matters are greatly complicated when the tremie is handled from a floating derrick, which lurches when the load is lifted and again when it is dropped. To reduce washing in filling the tremie, a cement sack, loosely filled with straw, will act as a plunger and force the water before it as the weight of the concrete forces it through the tube. The sack will usually float to the surface after passing through the pipe.

The third trouble will cause loss of cement from the charge and churning of the deposited concrete. The pipe must be "picked", or lifted, just enough to start the flow, which will rarely bring the lower end out of the soft concrete previously deposited.

A concrete with a small aggregate is handled much more easily through a tremie than one with the larger sized stone used in open-air mass work.

In the small dams, the entire layer of 6 ft. was carried across at once, but, in Piers III and IV, two lifts of 5 ft. each were made, and the reinforcement was placed between. For that part of the blanket of Pier IV under the old span, a bottom-dump bucket was used, which which was "telegraphed" (fleeted or drifted) in under from the fixed derrick to a line hung from the old bridge floor. It is a point much debated in work with a bucket, whether a top cover protects the concrete from wash, or causes more wash by creating a partial vacuum as the charge empties. At Cherry Street, two canvas flaps, weighted on the edge, covered the surface of the concrete as the bucket was submerged. Observation of the blankets after unwatering the dams showed the expected layer of laitance and silt, from ½ in. to 2 in. thick, in the low spots of the surface, and a decided unevenness of the top surface, in spite of the care taken to get it as level as possible. Concrete under water (due to being in a denser medium) stands in slopes much steeper than would be possible with that laid in air. Samples taken from the blankets, after cleaning the surface, showed that an excellent, dense concrete had been secured. Of under-water concrete

9 000 cu. yd. were required for all the piers, of which 7 050 cu. yd. had been placed at the end of the 1912 season.

The floating mixer plant previously referred to is a type of equipment, used by several contractors along the Great Lakes, utilizing the now familiar method of tower distribution, as developed for land work. (See Figs. 10 and 11.) This means of putting concrete into place has several advantages over the floating derrick and bucket. There is less danger of damage to the forms by knocking out braces or breaking wiring, the men do not have to keep out of the way of rapidly moving buckets, and the speed is practically limited only by the capacity of the mixer. For work in pier forms, a rate of 350 batches per 9-hour shift was commonly maintained. For shorter periods, speeds up to 50 batches per hour were made, but the total for the shift was usually cut down by the minor break-downs of material-handling machinery and delays incident to the shifting of sand and stone scows. The plant consisted of boilers, a hoisting engine, a stiff-leg derrick for hoisting material to the hopper bins, a $\frac{3}{4}$-cu. yd. Koehring mixer, and the elevator tower, all mounted on a barge 32 ft. wide and 100 ft. long. The tower was 56 ft. high, with the discharge hopper mounted on a slide, so that the chute could be raised or lowered readily, without changing its slope.

For Piers I, II, and IX, as previously mentioned, piling had been substituted for the concrete cylinders. The procedure in building these piers was much the same. After excavating inside the coffer-dam to a stiff blue clay, a test pile was driven, which showed the probable penetration that would be secured. Piling was then ordered of a length to give this penetration and extend a couple of feet above the bottom. In driving, a pipe of heavy 20-in. tubing was set in the leads, with its lower end on the bottom. The piles, which were usually shorter in length than the depth of the water, were dropped into this tube and driven with a follower to the desired penetration, by a No. 1 Vulcan steam hammer. About 25% of the piles in the two abutment piers had a batter of 2 in 12 and, for these, long piles were driven with batter leads and later cut off by a diver, or cut off above the blanket after pumping.

During the work on Pier II, an accident occurred in which loss of life was narrowly averted, and gave a remarkable proof of the abuse steel sheeting will stand. Before driving the piles, the channel had

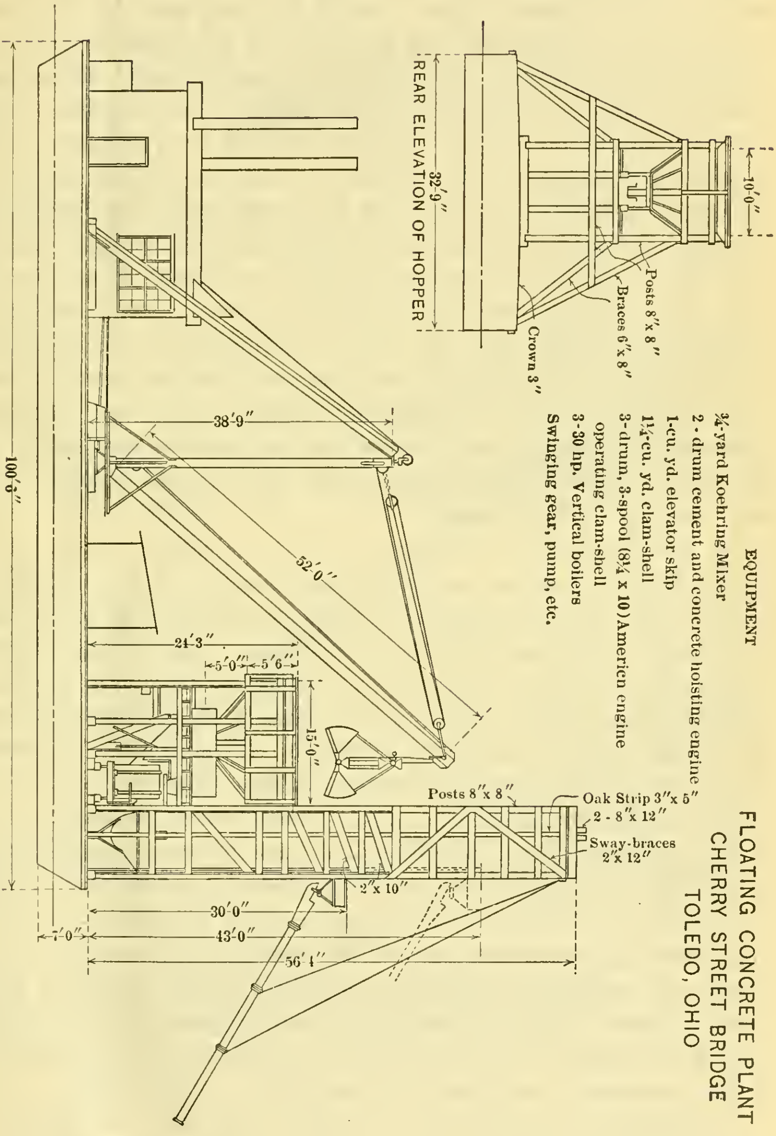

FLOATING CONCRETE PLANT
CHERRY STREET BRIDGE
TOLEDO, OHIO
EQUIPMENT
¾-yard Koehring Mixer
2 - drum cement and concrete hoisting engine
1-cu. yd. elevator skip
1¼-cu. yd. clam-shell
3-drum, 3-spool (8¼ x 10) Americn engine operating clam-shell
3-30 hp. Vertical boilers
Swinging gear, pump, etc.
REAR ELEVATION OF HOPPER
32′-9″
10′-0″
Posts 8″x 8″
Braces 6″x 8″
Crown 3″
38′-9″
52′-0″
24′-3″
5′-0″
5′6″
15′-0″
Posts 8″x 8″
Oak Strip 3″x 5″
2 - 8″x 12″
Sway-braces 2″x 12″
2″x 10″
30′0″
43′0″
56′4″
7′0″
100′0″

FIG. 10.

been dredged with a floating dipper-dredge to blue clay, at a depth of 30.4 ft., and then a 30 by 60-ft. coffer-dam was driven. The piling was of the fabricated channel and Z-bar type used by the contractor throughout the work, and was driven to an average penetration of 10 ft. in the blue clay. After completing the driving of the bearing piles, the blanket of concrete, 6 ft. in thickness, was deposited under water, as in the other piers, leaving a 24.5-ft. head of clear water to be pumped against. The timbering was built up in place and sunk to position, and the final wedging was done as the racks were unwatered. The dam proved to be tight, and all the leaks were readily stopped with the mixture of oak sawdust, manure, and ashes. It was found that more discharge had been provided than was needed, and some of the pumps were cut out. At 11 P. M. on June 28th, 1912, the water was down 18 ft.; 5 min. later, just after the pump men had left the hole for their lunch, the timbering suddenly gave way. Though unwitnessed, it seems most probable that the accident was caused by the jar and weight of the pumps forcing the transverse timbers from their bearings on the walings.

At first sight, the wreck (Figs. 12 and 13) seemed to be complete, but as this pier was now one of the critical points in the completion of the bridge at the promised date, it was decided to make an attempt to straighten the dam, re-timber, and pump. This hardly seemed feasible at first, as the interlock was expected to be sprung so much by the bending of the sheeting over the edge of the footing-block that the leakage would be impossible to control. Examination of the dam above water, and by a driver below, showed that, following the collapse of the bracing, the sheeting had been sharply bent at an angle of 40° over the concrete. From July 1st to July 12th, the gangs were engaged in pulling the walls back to line at the water's surface and sinking a second set of timbering, after removing the wreckage. On July 13th, pumping was started, and, to the surprise of all concerned, the water was lowered so fast that some of the pumps had to be shut down in order to allow the carpenters time to wedge up the racks as they were unwatered. By July 15th, the water was down, and the leakage was easily controlled by a single 6-in. Cameron pump.

Examination showed that the sheeting had withstood its severe punishment without damaging the interlock at any point, and was holding tight, though bent in what would have been thought to be

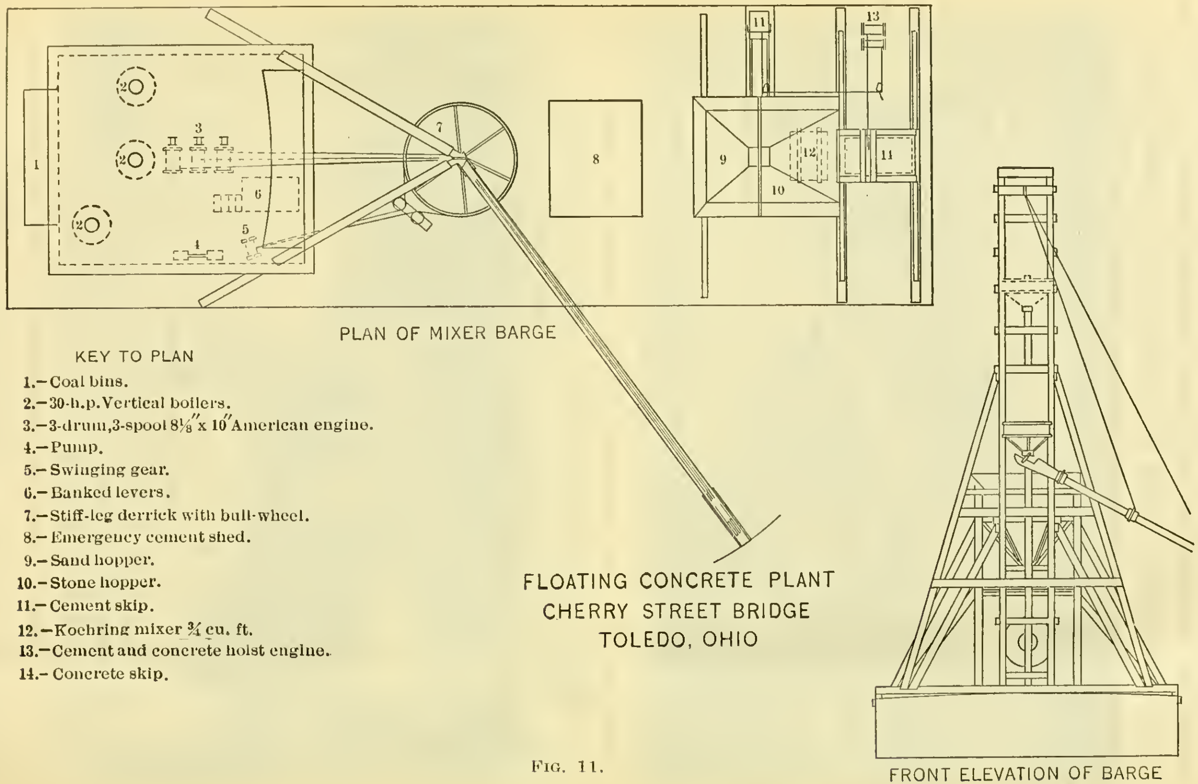

PLAN OF MIXER BARGE
KEY TO PLAN
1.—Coal bins.
2.—30-h.p. Vertical boilers.
3.—3-drum, 3-spool 8⅛" x 10" American engine.
4.—Pump.
5.—Swinging gear.
6.—Banked levers.
7.—Stiff-leg derrick with bull-wheel.
8.—Emergency cement shed.
9.—Sand hopper.
10.—Stone hopper.
11.—Cement skip.
12.—Koehring mixer ¾ cu. ft.
13.—Cement and concrete hoist engine.
14.—Concrete skip.
FLOATING CONCRETE PLANT
CHERRY STREET BRIDGE
TOLEDO, OHIO
FRONT ELEVATION OF BARGE

Fig. 11.

impossibly sharp curves. Forms were rapidly built, and the pier was completed to the water line by July 19th.

Throughout this job, the contractor used about 40 000 ft. of steel sheet-piling of this type, in nine coffer-dams of an aggregate area of 21 300 sq. ft. It was easily driven and pulled, though there was considerable difficulty at times in starting to pull the piling where the footing-block had been cast under water directly against the riveted sheeting. All of it was used at least twice, and much of it oftener than that, on this first half of the job. The dams were unwatered in all cases with very little difficulty, against heads of clear water of from 20 to 25 ft.

The first work above water was started in May, 1912, when the piers on the east side of the channel had all been raised to Elevation 92, and the placing of the arch centers was begun. The contractor's scheme for the construction of the arch ribs was to cast them in successive parallel strips of about 12 ft. each, supporting the forms on steel centering until the concrete had attained its strength, and then shifting the centers sideways to cast the adjacent section. This plan was followed, with some variations caused by the necessity of rushing work on the approach of cold weather at the end of the season.

The contractor used five three-hinged steel arches, of span lengths to fit the openings on the east side, for the arch centering. For the two 108-ft. arches on the west side, later, the 108-ft. and 100-ft. centers were brought across from the other side of the channel, and the shorter one was lengthened out by bolting on members at the crown. Each center consisted of two three-hinged arch trusses, with a temporary tie to take the thrust during erection. In service, the thrust from the concrete load is carried into the piers through large bent plates at the lower hinges, the plates also serving as forms for that part of the pier. (See Fig. 14.) Fig. 16 shows the details of the centering, with the lowering wedges and rollers. The centers were supported on I-beams which crossed the piers at the water line in slots left for that purpose. These were concreted into the work, and were to be cut off later back of the face of the pier. These I-beams carried the longitudinal beams on which the rollers traveled in shifting the centers laterally. Fig. 17 shows the details of the forms for the arch ring. To secure the proper curve of the arch intrados, the outlines of semi-arches were first laid out on a large wooden platform in the

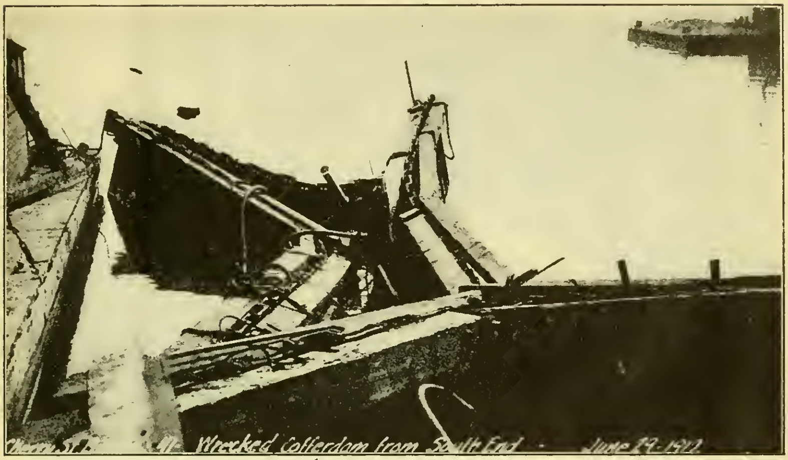


FIG. 12.—WRECKED COFFER-DAM OF PIER II, FROM SOUTH END.

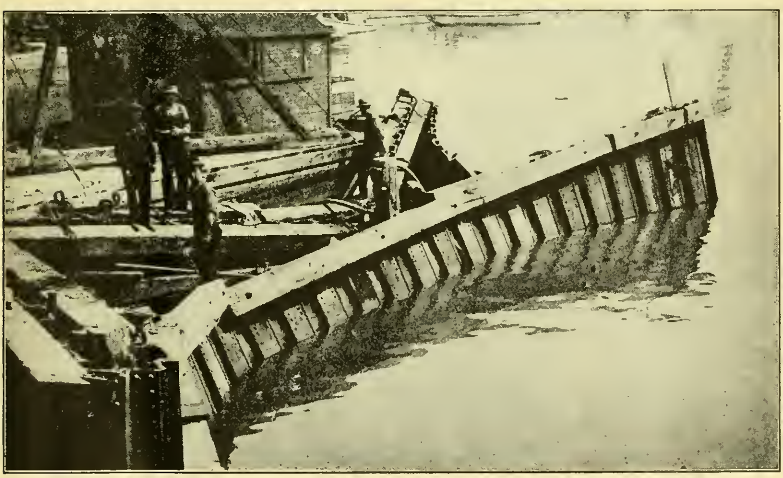


FIG. 13.—WRECKED COFFER-DAM OF PIER II BEFORE RE-TIMBERING AND RE-PUMPING.

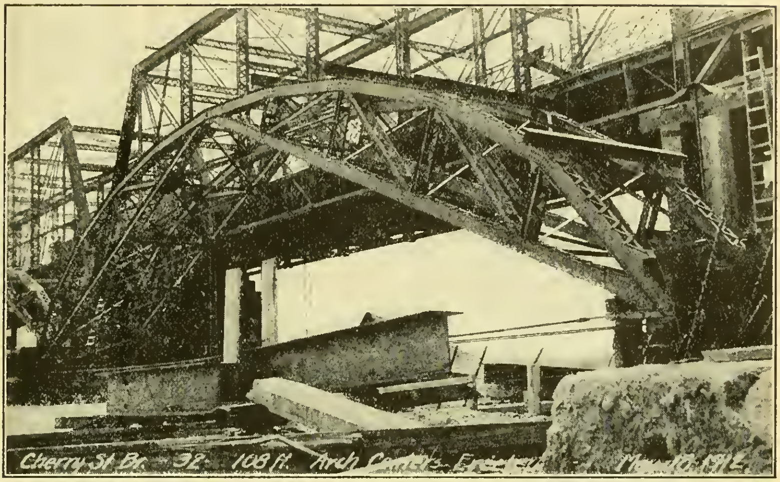


FIG. 14.—THREE-HINGED ARCH CENTERS FOR SUPPORT OF 108-FT. ARCH FORMS.

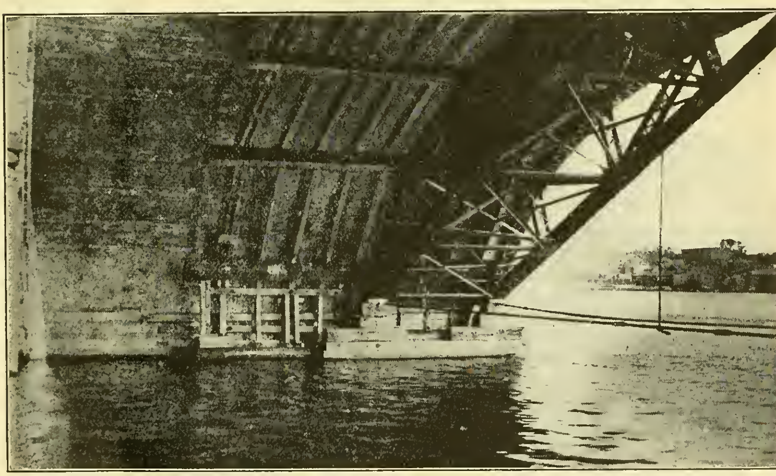


FIG. 15.—ARCH CENTERS IN PLACE READY TO COMPLETE HALF WIDTH OF ARCH IN ONE OPERATION, USING AUXILIARY NEEDLE-BEAMS.

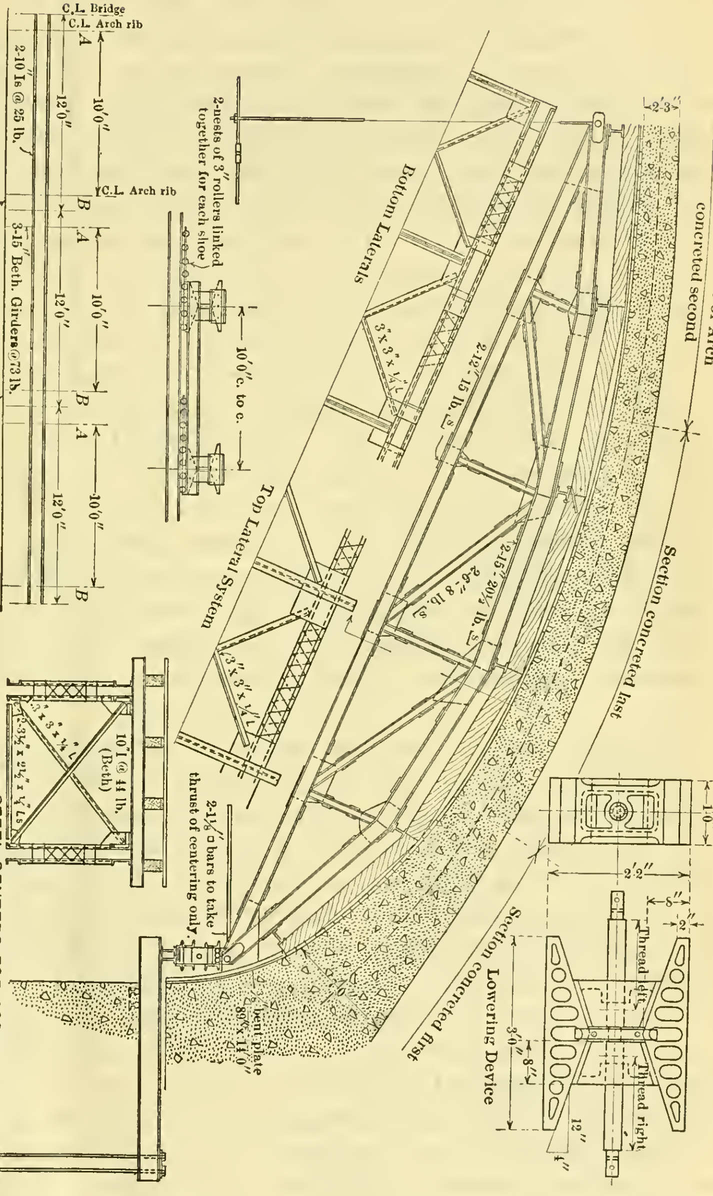


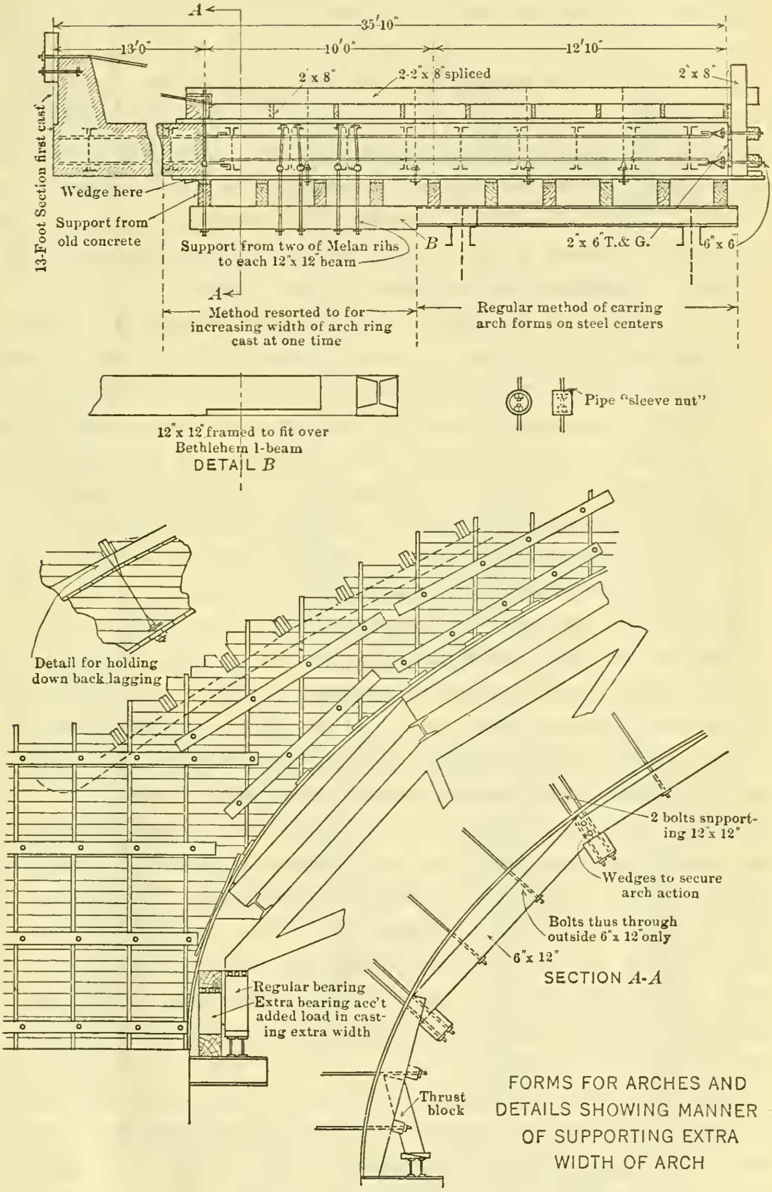
FIG. 16.

STEEL CENTERS FOR 108-FOOT ARCH CHERRY STREET BRIDGE, TOLEDO, OHIO.

yard, and then a section of arch center was placed on its side in the correct relative position and templates made, from which the 6 by 12-in. joists were cut to the proper curve by a band-saw in the shop.

When planning the programme for pouring the arch rings, it was seen that the use of the arched centers introduced a problem not common to the ordinary timber falsework. This was the unbalanced thrust which would come on any pier from a partly or entirely completed arch on one side only. It was decided to start with the smallest arch at the east end and carry the concrete out only one-third of the distance to the crown at each pier in succession from there to the channel. This was done, the concrete fleet being shifted from side to side of the arch, so as to place fresh concrete on each surface not later than 2 hours after leaving it. As three surfaces had to be kept fresh, this moving took time and caused some risk of having a surface set before getting back to it. This, in fact, occurred at one of the points, and the rapidly hardening concrete had to be removed to form a surface normal to the thrust before the concreting could be continued. After pouring the lower part of the arches in this way, the crowns were finished as soon as the first concrete had set enough to permit the removal of the bulkheads. The floating mixer and tower handled this work very satisfactorily, as the batches could be chuted to down-spouts placed where needed through the top lagging of the forms.

After completing the first 12-ft. section, it was realized that if the plan of casting the arch ring in 12-ft. sections was adhered to, the work on the first half of the bridge could not be completed that season. It was then decided that it would be possible to cast the remaining width of 23 ft. of the half-arch barrel at one time, by increasing the load on the centers somewhat and dividing the remainder of the load between the Melan rib reinforcement and the section of arch previously completed. How this was accomplished is shown in Figs. 15 and 17. The transverse beams carried by the steel centers were prolonged with 12 by 12-in. timbers, notched to fit over the beams at one end and supported at the other end by bolts from the old concrete and in the center by stirrups which threw part of the load on the latticed ribs. As it would take more time to concrete this increased width of arch ring, the order of pouring was changed, with the idea of reducing the number of surfaces to be kept fresh. In order to do this, each



A
35'10"
13'0"
10'0"
12'10"
2"x 8"
2-2"x 8" spliced
2"x 8"
13-Foot Section first cast
Wedge here
Support from old concrete
Support from two of Melan ribs to each 12"x 12" beam
B
2"x 6" T.& G.
6"x 6"
A
Method resorted to for increasing width of arch ring cast at one time
Regular method of carring arch forms on steel centers
12"x 12" framed to fit over Bethlehem I-beam
DETAIL B
Pipe "sleeve nut"
Detail for holding down back lagging
2 bolts supporting 12"x 12"
Wedges to secure arch action
Bolts thus through outside 6"x 12" only
6"x 12"
SECTION A-A
Regular bearing
Extra bearing acc't added load in casting extra width
Thrust block
FORMS FOR ARCHES AND DETAILS SHOWING MANNER OF SUPPORTING EXTRA WIDTH OF ARCH

FIG. 17.

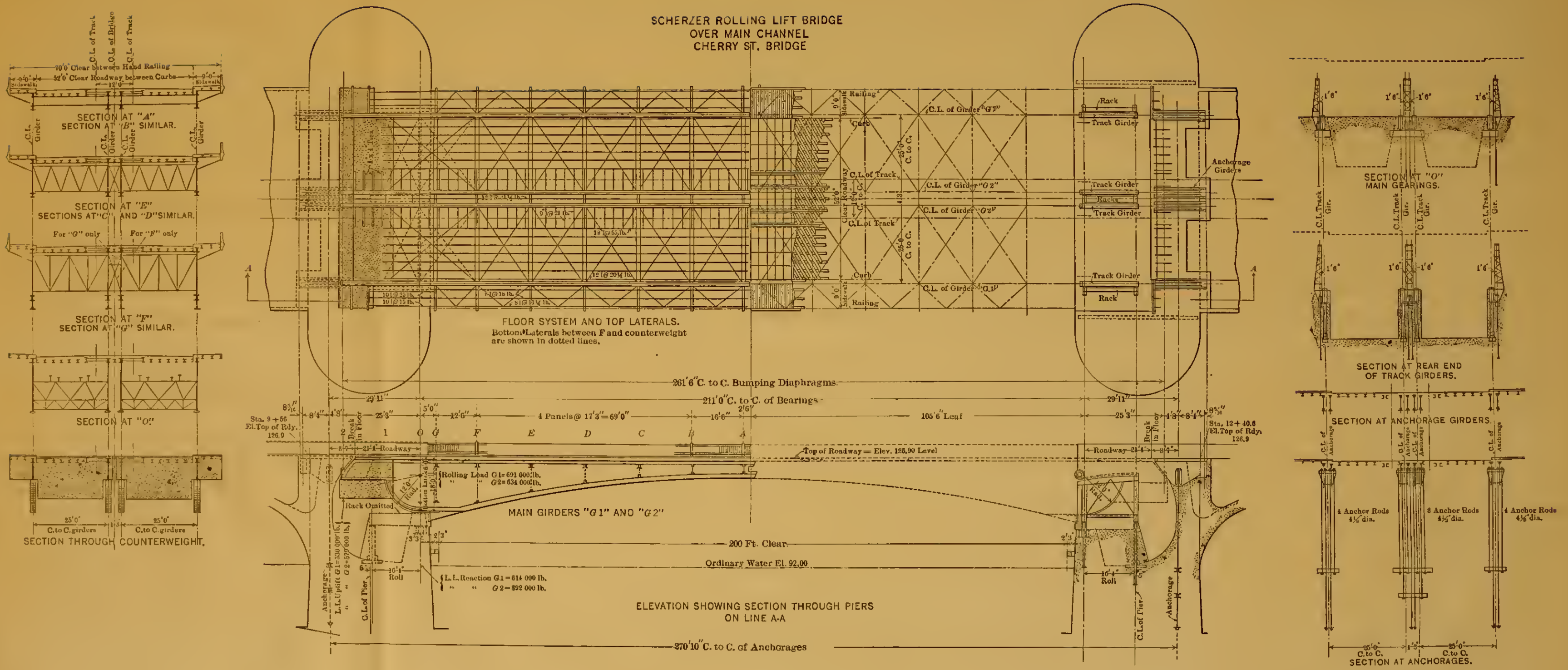
pier was poured separately up to the bulkheads at the springing line. This also embedded the ends of the latticed ribs and made them ready to act as arches in carrying their proportion of the load, later. As soon as the piers had set enough to resist the thrust of the arch centers, the haunches of the arches were poured to a point 15 or 20 ft. from the crown, the top lagging being in place before concreting was started. At this point, where the slope was about 2 in 12, the top lagging was stopped. While pouring the haunches, the crowns of the arches were loaded with concrete blocks, and, later, when pouring the crowns, the adjacent uncompleted arches were similarly loaded.

When concreting, one man worked between each pair of latticed ribs, puddling with his feet, or using a long 2 by 4-in. pole to reach the far edge of the concrete against the top lagging. Particular care was taken in this way to avoid the formation of rock pockets. The concrete came into the forms through hoppers placed at regular intervals along the top lagging. The plant was shifted from side to side of the arch at intervals of about 2 hours, so that fresh concrete was placed on the old before it had set more than 2½ hours. At this age, in the weather during which most of the work was done, the surface of the concrete was still soft enough to insure a good bond.

One of the chief difficulties encountered in the building of the spandrel walls, after the arch rings were complete, was the placing of the facing concrete which was used in order to secure an attractive texture of the concrete surface after bush-hammering. This mixture was of blue-gray and white limestone chips with a mortar of stone screenings and sand. Facing "irons" were first attempted, but were not feasible on account of the wet mixtures in use and a tendency to leave voids or rock pockets in the face of the work. The final method adopted was to use a liberal quantity of the mixture and pour it from buckets against the forms, keeping its level always slightly higher than the surface of the concrete. The rock pockets on the face that resulted from the attempt to use facing irons were filled later by use of the cement gun.

The arches were not ready for water-proofing until very late in the season, for the best results. The five-ply water-proofing over the arch barrels and inside of the spandrel wall, however, was placed as rapidly as possible after the completion of the concrete, so that the back-filling might be commenced. The order of work was to clean

PLATE XXXIX
PAPERS, AM. SOC. C. E.
OCTOBER, 1915.
CHASE ON
CHERRY STREET BRIDGE, TOLEDO, OHIO.



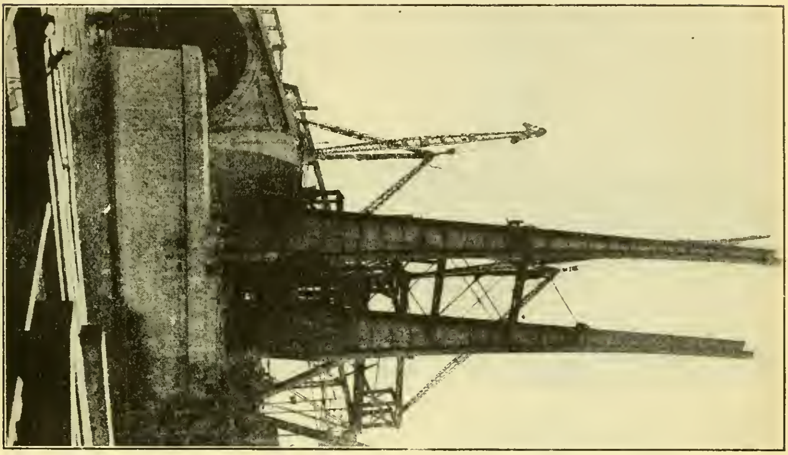


FIG. 18.—ERECTION OF FIRST HALF OF SCHERZER BASCULE IN OPEN POSITION, PIER IV.

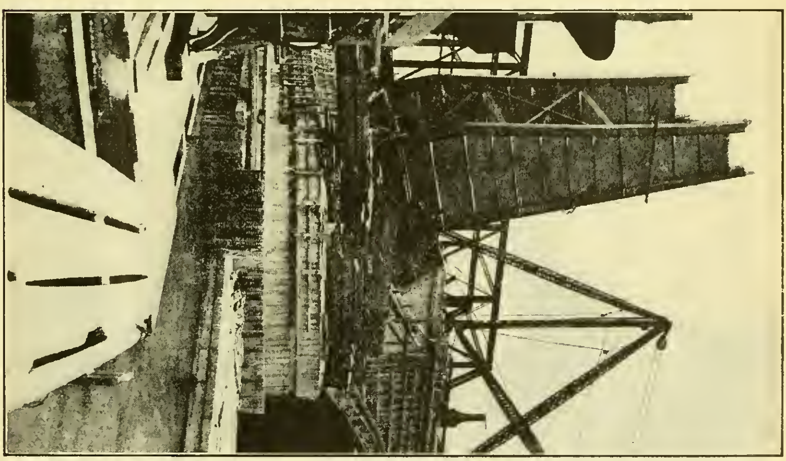


FIG. 19.—ERECTION OF BASCULE GIRDERS, PIER III.

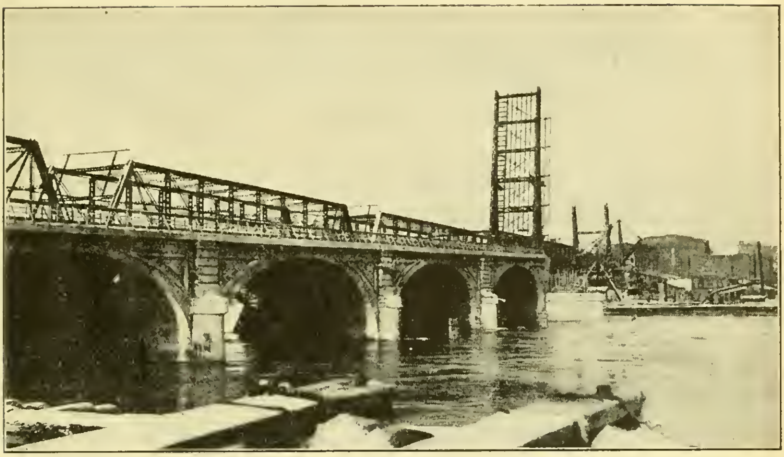


FIG. 20.—PROGRESS VIEW, WINTER, 1912-13, LOOKING WEST. FIRST HALF OF BRIDGE NEARLY COMPLETED, OLD BRIDGE STILL IN USE.

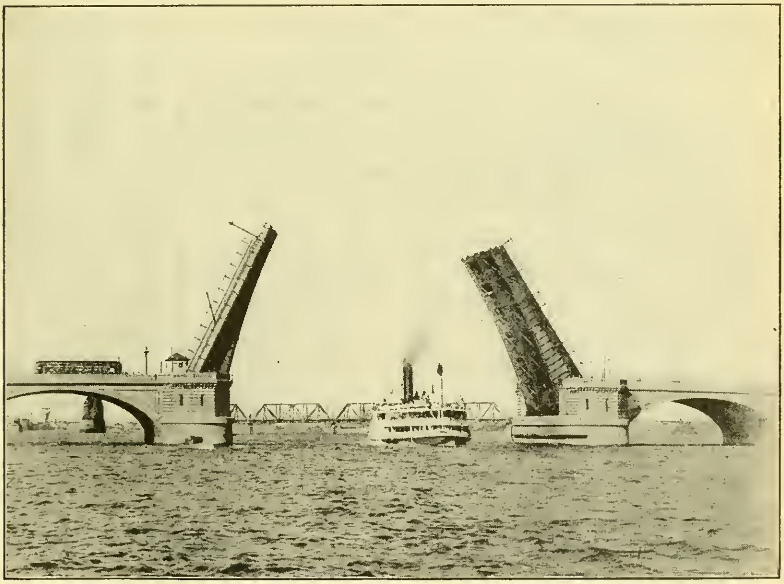


FIG. 21.—CHERRY STREET BASCULE SPAN IN OPERATION. PASSING A LAKE BOAT.

the concrete surface thoroughly and then mop it with pitch. A three-ply layer of felt was then built up, and over it a final course of two-ply was placed. A layer of pitch was mopped on between all felt, and a final heavy coating was applied to the completed membrane. Over the expansion joints in the wall, the felt was laid up over a stick, which, when pulled, left a fold in the water-proofing, reinforced by two strips of canvas.

For back-filling, sand and gravel were dredged from the river, just below the bridge, by floating derricks and dropped from the clam-shell buckets directly into place. The load was placed first mainly over the haunches along the entire series of arches, so that too much unbalanced load would not be thrown on the piers; it was then leveled off and shoveled to place by hand. Only material that would drain rapidly and easily was used.

Bascule Span.

The erection of the half leaves of the bascule steel was started in the fall of 1912, as soon as the back-fill had been placed so that the erection derricks could be set up. The steel was unloaded from a siding near the river and transferred to barges. The main girders were delivered in three sections, the largest of which weighed 36 tons. To handle these, the contractor used two of his standard steel derricks, with heavy cast-steel mast head, mast seat, and one-piece gooseneck. One derrick was elevated on bents and partly supported by the top chords of the old bridge, in which position it could place the top section of the girders. (See Fig. 18.) The other derrick was set directly on the spandrel filling, and was only used for the first heavy lifts. It was then dismantled and transferred to the other side, where it handled the large sections alone. (See Fig. 19.) Later, on that side, the boom was spliced out to 110 ft. to handle the top section of the girder and floor system.

Cold weather, high winds, and sleet and ice storms made the erection in winter very difficult, but it was pushed, in order to be sure of having the half width of the bridge ready for use at the break-up of the winter of 1912-13. (See Fig. 20.)

On the original designs for this bridge it had been determined that the movable span was to be a double-leaf bascule, supported by four lines of cantilever girders the lower flanges of which curved to

an arch. The clear opening is 200 ft. The official design was for a simple trunnion bascule, a general drawing of which has been published in the *Transactions* of this Society.* Bids were taken on this

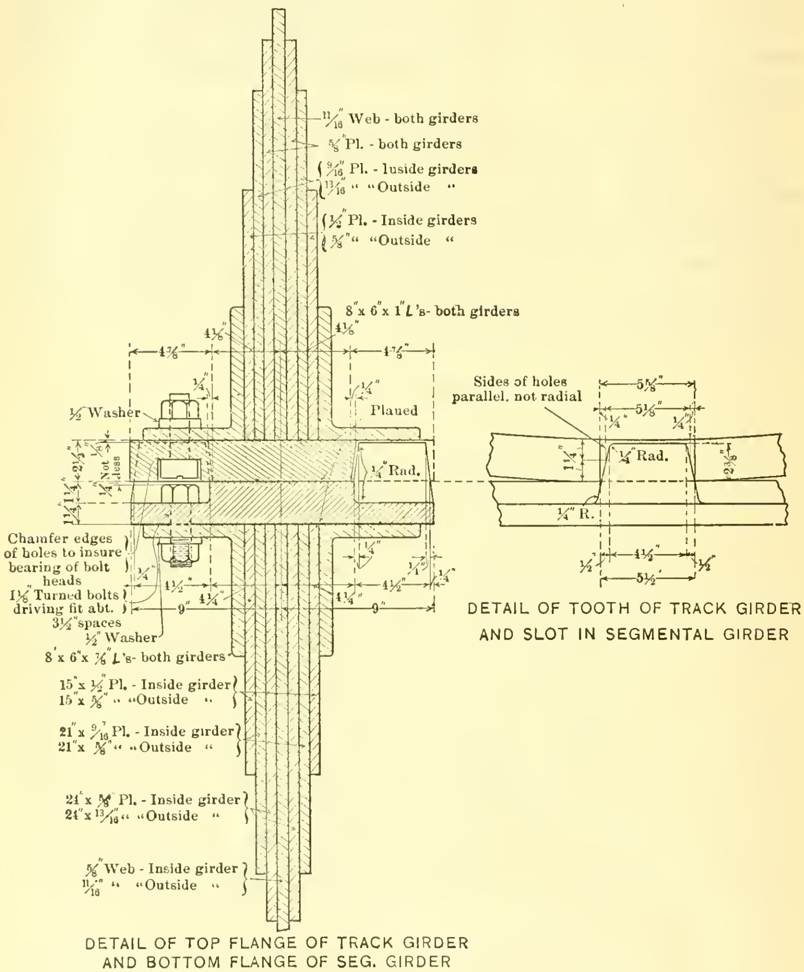
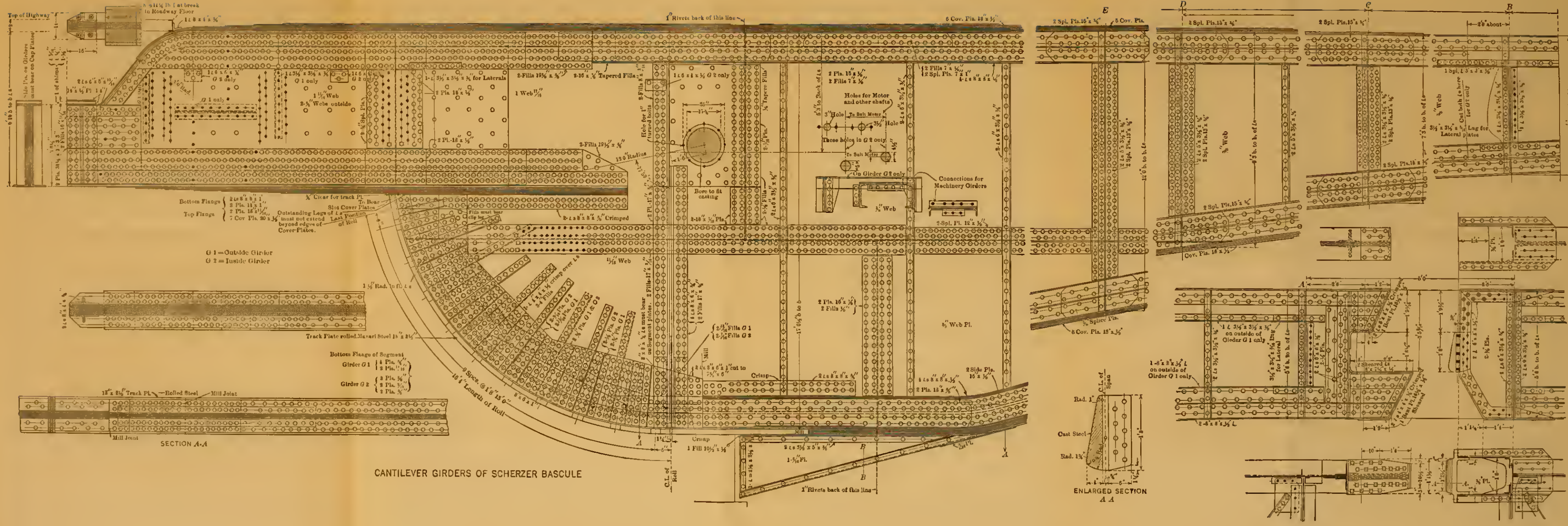


FIG. 22.

and alternate designs submitted by various movable bridge patentees. The Scherzer Rolling Lift Bridge Company was the successful bidder,

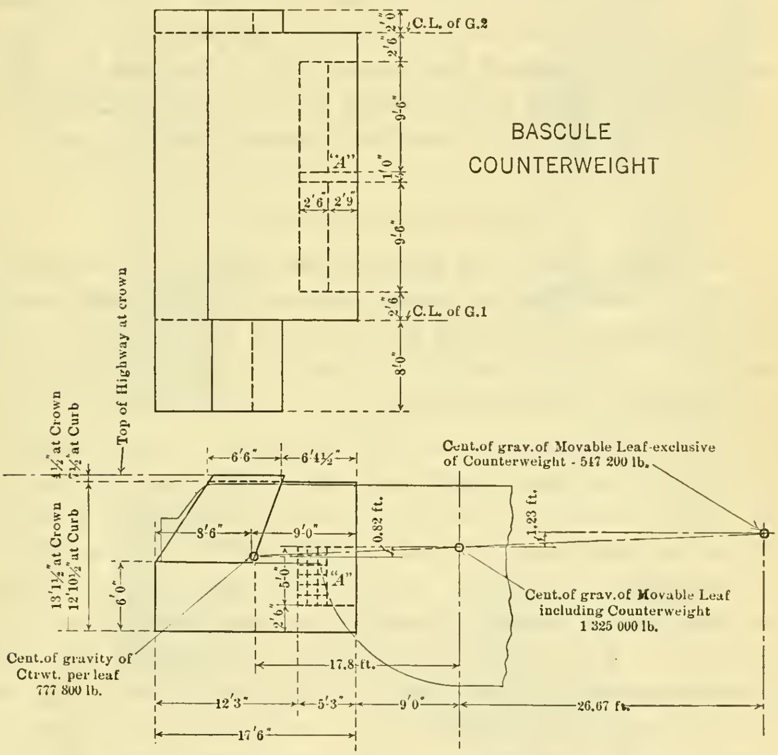
* Discussion by Wilbur J. Watson, M. Am. Soc. C. E., of "Movable Bridges", by C. C. Schneider, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LX (June, 1908), Plate XXXII.

PLATE XL.
PAPERS, AM. SOC. C. E.
OCTOBER, 1915.
CHASE ON
CHERRY STREET BRIDGE, TOLEDO, OHIO.



CANTILEVER GIRDERS OF SCHERZER BASCULE

and that Company designed the present bridge. Plate XXXIX shows the principal features of the structure. Each girder (Plate XL) is 131 ft. 6 in. long over all, of which 105 ft. 6 in. is the distance from point of support to center of span. The girders vary in depth from 17 ft. 0½ in. at the pier to 5 ft. 6 in. at the center. The rolling segment is shod with a Mayari (nickel-chrome) steel plate, 2½ in. thick, secured



Concrete counterweight to equal 777 800 lb. per leaf, averaging in place 147 lb. per cu. ft. This includes 34 900 lb. of concrete blocks, so distributed in pocket "A" as to balance the bridge properly in all positions.

FIG. 23.

to the flanges by turned bolts. (See Fig. 22.) The track plates (Plate XLI) are of cast steel. In the half leaf first erected the two girders are 25 ft. from center to center, and the inside one is 2 ft. 1½ in. from the center line of the bridge, making possible the erection of the second half of the bridge while the first is in use. To counter-

balance the dead load of each girder, 387 tons of concrete counterweight is provided, as shown by Plate XXXIX and Fig. 23. The live-load uplift at the rear end of each girder is transferred to four round steel rods extending to an anchorage in the masonry of the pier. Details of the bascule floor are shown on Plate XLII.

A 75-h.p., railway-type motor is mounted between each pair of girders to operate the structure. From this motor, a train of gears reduces the speed from 560 to 1.1 rev. per min. at the drive pinion. Each leaf can be operated from its own side of the channel, or both can be worked together by the operator in the main control house, the successive positions of both leaves being indicated there by signals.

Completion of the Work.

This account carries the history of the bridge up to the spring of 1913, at which time the writer's connection with the work ceased. The construction work in the season of 1913 was practically in all details a duplication of that done previously, after the removal of the old steel spans permitted building the second half of the structure. With the old bridge out of the way, and profiting by the experience of the preceding years of work and with an organization trained to the methods in use, far better time was made in this working season. Except that both halves of Piers III, IV, and VI, were complete at the beginning of the season, practically the same amount of work was done between March and December 31st, 1913, as in the preceding 2½ years. To avoid the loss of time incident to casting the arch rings in parallel sections, an adaptation of the method followed during the preceding year, to increase the width of strip cast, was used, together with some pile falsework, to permit of casting the entire remaining width of the arch ring at one time. Erection of the bascule was started before the end of the year, and by January 1st, 1914, the work of the contractor for the concrete construction was practically done and most of the bascule steel was in place. The fact that the channel was closed by ice allowed the erection sub-contractor to place the girders for the second half of the span in their horizontal position and connect them to the first half leaf while it remained closed.

PLATE XLI.
PAPERS, AM. SOC. C. E.
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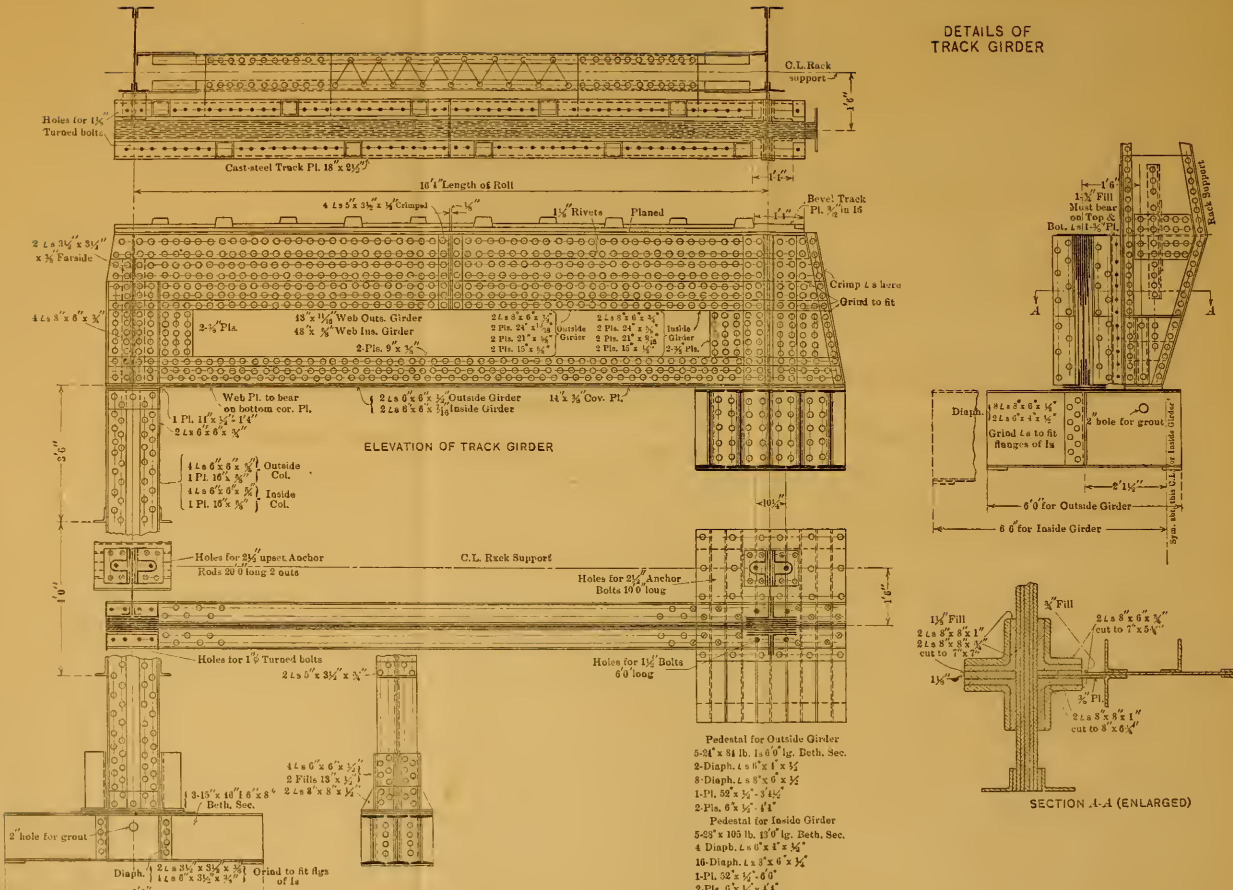
DETAILS OF TRACK GIRDER



Quantities and Cost.

The unit prices bid by the C. H. Fath and Son Construction Company, on March 3d, 1910, for this work were as follows:

Removing old masonry from piers and old rip-rap from ice-breakers and around old piers, per cubic yard	$1.50
Removing timber grillage from under old piers, per 1 000 ft. b. m.	15.00
Pulling old piles, each	3.00
Excavation and removal of sand, earth, and gravel inside coffer-dams and above cylinder piers, per cubic yard	1.50
Clay in coffer-dams or in cylinders, per cubic yard	1.65
Hardpan in cylinders, per cubic yard	1.80
Rock in cylinders, per cubic yard	3.50
Steel rings in cylinder shafts (not removed*), per pound	0.07
Steel rings in cylinder shafts (removed*), per pound	0.04
Timber lagging for cylinder shafts, in place, per 1 000 ft. b. m.	70.00
Old rails for reinforcing piers, abutments and cylinders, per long ton	30.00

Class "AA" concrete, for slabs, beams, girders, columns, etc.	1:2:4 mix	$10.80 per cu. yd.
Class "A" concrete, for arch rings	1:2:4 mix	19.00 " " "
Class "B" concrete, for spandrel walls, etc.	1:3:6 mix	10.80 " " "

(In construction this was actually changed to a 1:2:4 mix, using small stone instead of the 2½-in. limit specified.)

Class "C" concrete for piers above mean water level	1:3½:7 mix	$7.35 per cu. yd.
Class "D" concrete for mass work in piers below mean water level (with embedded stone permitted up to 33%)	1:3½:7 mix	6.00 " " "
For the same, in cylinders		5.25 " " "

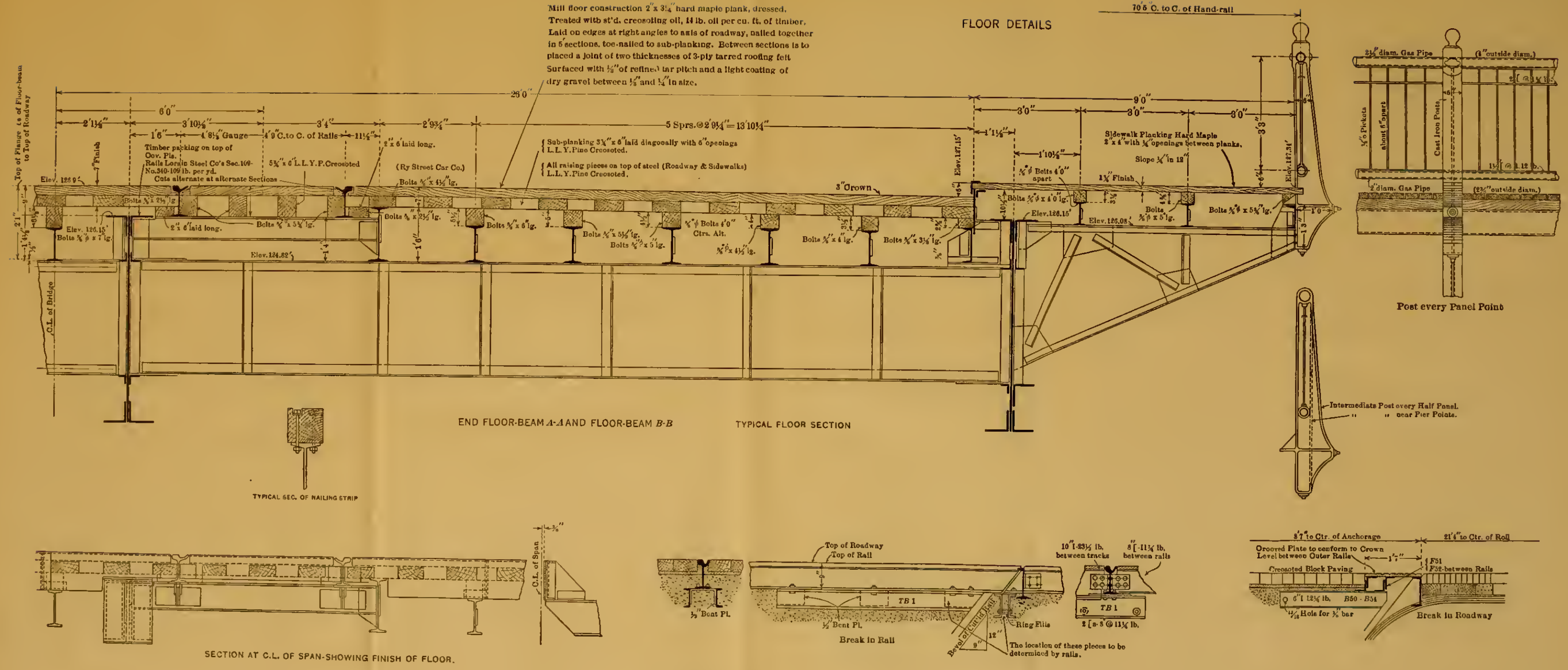
* * *

* Provision was made for using metal lining for the cylinders, if necessary, at a price of 7 cents per lb. if left in place. When the method of foundation was changed, this price was applied to the steel cylinders, with a deduction of 4 cents per lb. for steel removed.

To January 1st, 1914, the amount expended for various items in the construction of this bridge was as follows:

Maintenance of old bridge and removal of old masonry, etc.		$42 196.80
Construction and maintenance of the nine coffer-dams, lump sum		181 500.00
Excavation, under four classifications (with 75 cu. yd. of rock)	31 625 cu. yd.	49 075.05
Timber piling, in place	35 779 lin. ft.	14 311.60
Steel and wood lagging for foundation cylinders		103 595.82
Mass concrete in cylinders, piers, etc., in three classifications	19 795 cu. yd.	119 596.05
Concrete in arch rings, 1 : 2 : 4 mix	8 916.6 cu. yd.	177 440.34
Concrete in spandrel wall, pier faces, and viaduct, in three classifications	7 519 cu. yd.	88 705.00
Concrete in foundation blankets, placed under water, including cost of extra cement	9 420.0 cu.yd.	63 024.30
Concrete parapet wall, total lump sum = $10 000		7 800.00
Finishing surface of bridge, total lump sum = $6 000		3 000.00
Steel reinforcement	732 tons	54 210.00
Water-proofing arches and spandrel walls, lump sum		20 000.00
Spandrel filling	11 800 cu. yd.	9 440.00
Setting bascule anchorage	197.5 tons	5 925.00
Miscellaneous items		3 785.00
		$943 605.16
Contract cost of bascule		132 980.00
		$1 076 585.16
All other expenses, May, 1906, to January 1st, 1914		89 082.23
Total cost to January 1st, 1914		$1 165 667.39

PLATE XLII.
PAPERS, AM. SOC. C. E.
OCTOBER, 1915.
CHASE ON
CHERRY STREET BRIDGE, TOLEDO, OHIO



These items may be roughly summarized as follows:

Substructure work, new	$518 000	
Removal of old bridge masonry, etc	32 000	
		$550 000
Concrete superstructure	$380 000	
Steel bascule span	138 000	
		518 000
Miscellaneous expense		98 000
		$1 166 000

It has been estimated by those in charge of the work since January, 1914, that the cost to complete all unfinished work after that date, exclusive of the towers (the fate of which at present hangs in the balance) would be about $15 000, besides claims not yet adjusted amounting to about $40 000.

The original design of the Cherry Street Bridge was made by the Osborn Engineering Company, of Cleveland, Ohio, under the direction of Wilbur J. Watson, M. Am. Soc. C. E., at that time the Bridge Engineer of that Company.

In July, 1910, Ralph Modjeski, M. Am. Soc. C. E., was appointed Consulting Engineer in Charge. No changes were made by him in the general features of the design, with the exception of those involved in the methods of securing the foundations, already described, certain revisions in the proportions of the arch rings and the design of the reinforced concrete viaduct, and those changes incident to the use of the Scherzer instead of the trunnion bascule. All construction work was done under his supervision. In April, 1911, Mr. Arnold W. Brunner was retained by the City as Consulting Architect, and the external treatment of the structure was handled by him, in conjunction with Mr. Modjeski.

Gustavus A. Gessner, M. Am. Soc. C. E., was Resident Engineer from the commencement of work until December, 1911, and W. R. Weidman, M. Am. Soc. C. E., from that date until January, 1914. Mr. M. B. Case and the writer were Mr. Weidman's Assistants during the year and a half preceding June, 1913. The main contract, except for the bascule span, was in the hands of the C. H. Fath and Son Con-

struction Company, until the season of 1913, when the National Foundation and Engineering Company was organized to complete the work. Mr. M. J. Comer was Superintendent. The contract for the Scherzer span was assigned to the Toledo Bridge and Crane Company, and the steel was erected by the Kettler-Elliott Erection Company.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

COMPUTING RUN-OFF FROM RAINFALL AND OTHER PHYSICAL DATA

Discussion.*

By Messrs. W. G. Hoyt, and Adolph F. Meyer.†

W. G. Hoyt,‡ Assoc. M. Am. Soc. C. E. (by letter).—After making a careful study of the method of determining run-off from rainfall and other physical data, and after computing the "precipitation minus loss" and comparing it with the run-off of several Wisconsin streams, the writer feels that Mr. Meyer's method is not only rational, but is the best one devised to date. This paper gives convincing evidence of the great length of time spent in the study and computations. The writer welcomes a method which does not involve the determination of a factor for use in a formula of which precipitation is one factor and run-off is the result, but which attempts, instead, to ascertain the actual losses from precipitation, in order to determine the residual or run-off. Mr. Hoyt.

Of the prior methods devised to compute run-off, the writer believes that that of Vermeule§ is the most rational, because, in it he attempted to do roughly what Mr. Meyer has apparently done in detail. The writer hopes that other engineers will be sufficiently interested to measure and compute the flow of rivers in different parts of the country and compare the results, in order, not only to decide the accuracy of the method, but to ascertain what corrections and additions are needed. The writer feels that he cannot justly criticise Mr.

* Discussion of the paper by Adolph F. Meyer, M. Am. Soc. C. E., continued from September, 1915, *Proceedings*.

† Author's closure.

‡ Madison, Wis.

§ "Water Supply of New Jersey," 1894; and Annual Report, State Geologist of New Jersey, 1899.

Mr. Hoyt. Meyer's paper without comparing computed flow and measured flow at a number of stations.

In his introductory paragraph Mr. Meyer states that "there is need for a method of computing run-off from other physical data for the purpose of extending and supplementing short-term stream-flow records." It should be noted that he does not suggest that his method be used to compute the run-off of streams for which there are no records of observed run-off. To the layman the tables of run-off computed by Mr. Meyer's method would seem to be all that could be desired, whereas, without coefficients carefully derived from observational data, those tables might be far from accurate. Many expensive engineering works, or their remains, found in various parts of the country represent misplaced confidence in engineers or others who have thought it possible to predict stream flow with few or no base data.

At the beginning of this discussion the writer wishes to quote from the introduction to recent papers on the surface water supply of the United States published by the United States Geological Survey:*

"Even though the monthly means for any station may represent with a high degree of accuracy the quantity of water flowing past the gage, the figures showing discharge per square mile and depth of run-off in inches may be subject to gross errors, which result from including in the measured drainage area large noncontributing districts or omitting estimates of water diverted for irrigation or other use. 'Second-feet per square mile' and 'run-off (depth in inches)' have therefore not been computed for streams draining areas in which the annual rainfall is less than 20 inches, nor for streams in which the precipitation exceeds 20 inches if such computations might probably be uncertain and misleading because of the presence of large noncontributing districts in the measured drainage area, of omitting estimates of water diverted for irrigation or other use, or of artificial control or unusual natural control of the flow of the river above the gaging station. All values of 'second-feet per square mile' and 'run-off (depth in inches)' previously published by the United States Geological Survey should be used with extreme caution and such values in this report should be used with care because of possible inherent sources of error not known."

On account of such possible errors in the determination of "run-off (depth in inches)" from the drainage basin, it should be noted that these figures have been omitted where the Survey was reasonably sure that the entire basin was not at all times a contributor to the run-off. In certain sections of Minnesota, especially the western and northwestern parts, and also in northern Wisconsin, there are large swamp areas which contribute to the run-off of the streams only in times of high water. Such swamps and other non-contributing areas

* U. S. Geol. Survey, Water-Supply Paper 353, p. 15.

have considerable bearing on the determination of the coefficients, and any engineer attempting to use Mr. Meyer's method should study the drainage area very carefully. The writer, due to the fact that he was not well enough acquainted with the details of the method, did not base his coefficients so much on a previous study of the area as on the results he wished to obtain. Mr. Hoyt.

Mr. Meyer's method determines the "precipitation minus total losses, in inches," which, over a long period of years, approximately represents the run-off, if his coefficients based on the observed run-off are correct. As many of the published figures of run-off per square mile are subject to gross errors, they should be used with extreme care, and in determining a coefficient there should be a recomputation from the mean monthly flow and the actual contributing area, rather than the total area of the drainage basin above the gauging station.

It should be noted, also, that the records of stream flow presented in recent water-supply papers cover a "water year," beginning October 1st and ending September 30th, instead of the calendar year. The fact that the water year included by these dates is not strictly applicable throughout the entire country, and may even vary in the same locality from year to year, is of course recognized. No doubt exists as to the immense quantity of seepage flow in the central northwestern part of the United States, and for this reason Mr. Meyer's method of comparing the run-off during the period from March 1st of any year to February 28th or 29th of the following year, with the precipitation that occurs from November 1st of the preceding year to October 31st of the given year, is logical; it will be noted, however, that for some of the Wisconsin streams the computed run-off from November 1st of the preceding year to October 31st of the given year agrees more closely with the observed run-off for a similar period than the results shown in the summary of data and computations for the various drainage basins, most of which represent the run-off for the year beginning March 1st of a given year and ending February 28th or 29th of the following year. This is especially true of the Upper Wisconsin River, and is no doubt due to the large surface run-off during the period, November to February, inclusive.

As to the errors in the data on precipitation, much could be said. Undoubtedly many of the United States Weather Bureau Stations are seldom visited, and receive little supervision from the local offices. Considering the small area of the rain gauge and the large area over which the records apply, the estimates of monthly rainfall at a number of different stations in a drainage area agree surprisingly well. If the error introduced by the use of such records is at all constant, it is provided for in deriving the coefficients. Though the location of many Weather Bureau stations with respect to buildings and trees is unfavorable for the determination of the precipitation in the

Mr. Hoyt. surrounding country, it is doubtful if they are often moved, so that the error from year to year should be nearly the same.

In connection with his study of evaporation, Mr. Meyer has prepared a curve, Fig. 8, on which he has plotted the mean observed evaporation at University, N. Dak., the mean observed evaporation at Grand River Lock, Wis., and the mean computed evaporation at St. Paul, Minn. It is noted that the results of studies of evaporation records at Madison and Menasha, Wis., and at Iowa City, Iowa,* did not plot on this curve. As Mr. Meyer had access to these records, the writer would like to know why he did not use them in his determination of the evaporation, or at least plot the values on his curve. His evaporation curve from land and water is approximately the same as the formula used by FitzGerald and others, and the writer is somewhat surprised to find that this formula gives results so comparable with observed results, especially those taken at Mount Hope, N. Y., and Boston, Mass.

In the ordinary drainage basin, however, there is so little water surface that it is seldom necessary to take into account the loss from evaporation from such surfaces, but apparently this curve enters into Mr. Meyer's main curve for determining evaporation from land areas for various temperatures and rates of rainfall. The writer would like more detailed information in regard to the development of this curve; he would also like information as to the basis for the many assumptions that apparently had to be made.

The writer is of the opinion that run-off from rain falling in short intervals will differ radically from the run-off from the same quantity of rain well distributed over the month. Naturally, Mr. Meyer could not develop a curve that would take into consideration all these conditions, and it is supposed that his curve represents ordinary conditions of well-distributed precipitation.

No experiments have given conclusive information concerning actual losses by transpiration, and apparently Mr. Meyer's base curve is founded to a large extent on Van't Hoff's law. A loss determined by this curve, based on Van't Hoff's law and multiplied by a coefficient necessary to reduce it to what is called the normal seasonal transpiration, is as accurate as any. For a time the writer could not see why the losses from transpiration and evaporation could not be grouped into one loss to which one coefficient could be applied, but, as such losses may be accurately determined by future experiments, the method of using two coefficients is undoubtedly better.

The writer regrets that Mr. Meyer has not given, for each drainage basin for which he has determined the run-off, the coefficient he used to reduce the values of transpiration obtained from the transpiration

* "Water Resources of Minnesota," 1909–1912, State Drainage Commission, pp. 555–564.

curve to the so-called transpiration loss, and the evaporation from land surface obtained from the evaporation curve to the so-called evaporation loss from land surface. (The writer has used the word "so-called" in this connection because the quantities as determined from the evaporation and transpiration curves may not of necessity be strictly evaporation and transpiration losses.) It is hoped that Mr. Meyer will do this in his closing discussion, in order that those interested may see at a glance the relation between these coefficients and the extent to which they vary from year to year. Undoubtedly the accuracy of the method depends, not only on the accuracy of the coefficient, but also on the applicability of the same coefficient year after year, or so long as there are no radical changes in the drainage basin, such as marked deforestation, or the construction of large storage reservoirs. Apparently there is no reason why the coefficient should vary to any extent, but the writer is of the opinion that more than 2 or 3 years of records are necessary for its determination, as in order to use the data of 1 or 2 years' run-off it is necessary to make a careful study to ascertain whether during the year there was considerable seepage flow from the preceding year, or whether a large quantity of water was absorbed to make up for low ground-water during preceding years. Mr. Hoyt.

In order to determine approximately the variation of coefficients in different drainage areas, and the accuracy of the method, the writer has applied Mr. Meyer's method to several drainage basins in Wisconsin. These data and computations, together with a brief description of the characteristics of each of the basins, form part of this discussion. (See Tables 38 to 48.)

The coefficients used in the tables to reduce the values taken from the curve to those which would give the most consistent results are given in Table 36.

TABLE 36.

Drainage basin.	Coefficient.	
	Transpiration.	Evaporation.
Rock River Basin above Rockton, Ill	0.75	1.05
Black River Basin above Neillsville, Wis	0.70	0.88
Wisconsin River Basin above Rhinelander, Wis	0.62	0.59
Wisconsin River Basin above Merrill, Wis	0.66	0.66
Wisconsin River Basin between Merrill and Rhinelander, Wis	0.66	0.58

Provided the values as taken from the evaporation and transpiration curves for the year were added together, it would have been necessary to use the coefficients given in Table 37 to reduce the quantity (obtained by subtracting the sum of the evaporation and transpiration losses

Mr. Hoyt. from the precipitation) to a figure which would be consistent with the observed run-off.

TABLE 37.

	Mean.
Rock River Basin above Rockton, Ill.	0.98, varies from 0.85 to 1.31
Rock River Basin above Neillsville, Wis.	0.83, " " 0.73 to 1.02
Wisconsin River Basin above Rhinelander, Wis.	0.64, " " 0.42 to 0.76
Wisconsin River Basin above Merrill, Wis.	0.63, " " 0.49 to 0.72
Wisconsin River Basin between Merrill and Rhinelander, Wis.	0.64, " " 0.53 to 0.72

Mr. Meyer states, on page 593:*

"To the values of evaporation, in inches of depth per month, as taken off the curve, a coefficient must be applied to reduce these quantities to actual evaporation from the given water-shed. This coefficient ranges from about 0.95 to 1.25 for most water-sheds of the Northwest, and for similar ones elsewhere. The coefficient to be used depends on topography, vegetal cover, soil, subsoil, humidity, and wind. An extremely high coefficient of evaporation would result from flat topography devoid of vegetation, moderately pervious, shallow soil underlain with impervious subsoil or rock, low humidity, and high wind velocity. An extremely low coefficient would result from rugged topography, bare scanty soil underlain with rock, high humidity, and low wind velocity. Between these extremes the usual working values will be found. With a little experience, one can select coefficients for different water-sheds with considerable accuracy."

It should be noted that the writer finds an apparent range in coefficients from 0.58 to 1.05, instead of from 0.95 to 1.25. The large range in coefficient for such a small territory as Wisconsin is very unfortunate, but perhaps Mr. Meyer, by reason of his long use of the method, may be able to discover the reason for the large variation in coefficients obtained by the writer, or can find causative errors in the computation. The large variation in coefficients for the different basins shows the necessity of basing the coefficient on actual determinations of flow, and the range of values of the coefficients for the same basin emphasizes the need for records covering several years in order to obtain a mean coefficient that will eliminate the effects of seepage and storage. The run-off from the area between Rhinelander and Merrill was computed to ascertain if there was any radical change in coefficient due to the exclusion of the large part of the drainage area above Rhinelander that is controlled by reservoirs. Apparently, the operation of these reservoirs did not materially affect the total run-off, although its distribution has undoubtedly been changed. The writer has not attempted to compute monthly run-off for any of the streams, as he

* *Proceedings*, Am. Soc. C. E., for March, 1915.

feels that any computation of the monthly run-off by using the curves must be largely a matter of judgment in adding to and subtracting from the "precipitation minus losses" as determined from the various curves. It should be noted that Mr. Meyer gave no information concerning the construction of his curves for the flow of the Root River at Houston, Minn., to aid in the determination of surface run-off resulting from a monthly curve of "precipitation minus losses." Mr. Meyer would add to his already very valuable paper by giving detailed information relative to the development of these curves. In order to show the relation between the run-off computed from "precipitation Mr. Hoyt.

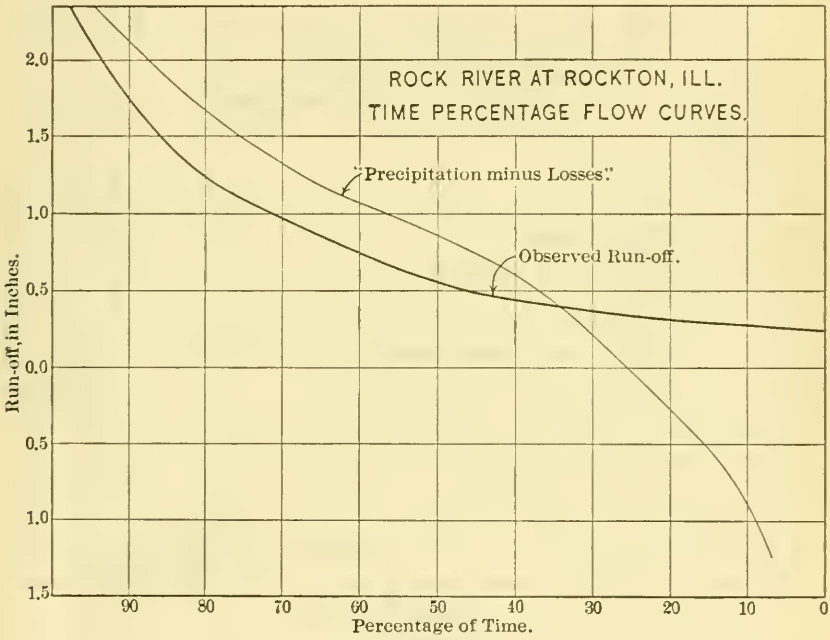


FIG. 38.

minus losses" and the actual run-off, curves showing the percentage of time covered by actual and computed records of run-off in inches for Rock River at Rockton, Ill., Fig. 38, Black River at Neillsville, Wis., Fig. 39, and Wisconsin River above Rhinelander, Wis., Fig. 40, are presented herewith.

These curves are the same as Mr. Meyer's frequency curves of run-off, Fig. 33. Apparently his frequency curves represent observed run-off and not "precipitation minus losses". It will be noticed that where there is very little ground storage, as on Black River at Neillsville, Wis., the percentage of time in which the actual run-off

Mr. Hoyt. is less than that computed from "precipitation minus total losses" is greater than where there is large storage, either natural, as on Rock River at Rockton, Ill., or natural and artificial, as on Wisconsin River at Rhinelander, Wis. For the station on Black River at Neillsville, Wis., the observed run-off is less than that computed from "precipitation minus losses" for about 70% of the time; on Rock River at Rockton, Ill., the observed run-off is less than the computed run-off for 66% of the time; and on Wisconsin River at Rhinelander, Wis., where there is large natural and artificial storage, the observed run-off is less than the computed for only 34% of the time.

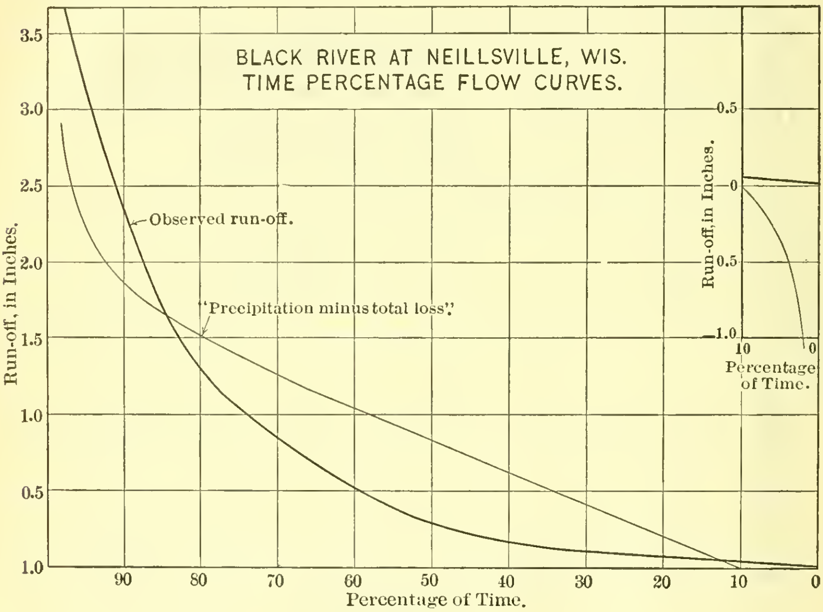


FIG. 39.

Until a very detailed study of a drainage basin has been made, it is doubtful whether the run-off computed from "percipitation minus losses" for any particular month can be translated into the observed run-off for that particular month with any great degree of accuracy. The writer realizes that Mr. Meyer's determination of the monthly flow of Root River in Minnesota checks remarkably close with the observed flow, so close in fact that he has been able to show the writer, under whose direction this gauging station has been maintained, errors in the observed flow. Knowing the characteristics of the flow of Root River as the writer does, he realizes that the determination of the

monthly flow of any river may be possible. He desires, however, more proof before his doubt changes to belief. Mr. Hoyt.

For this reason the writer prepared the curves on Figs. 38 to 40 in order to ascertain what results could be obtained mathematically without making any adjustments for ground storage or seepage flow. The tables show in general, that the observed run-off in any one month is larger than that computed from "precipitation minus losses," at least for Wisconsin streams, from about March 1st to August 31st, and that the run-off is less than "precipitation minus

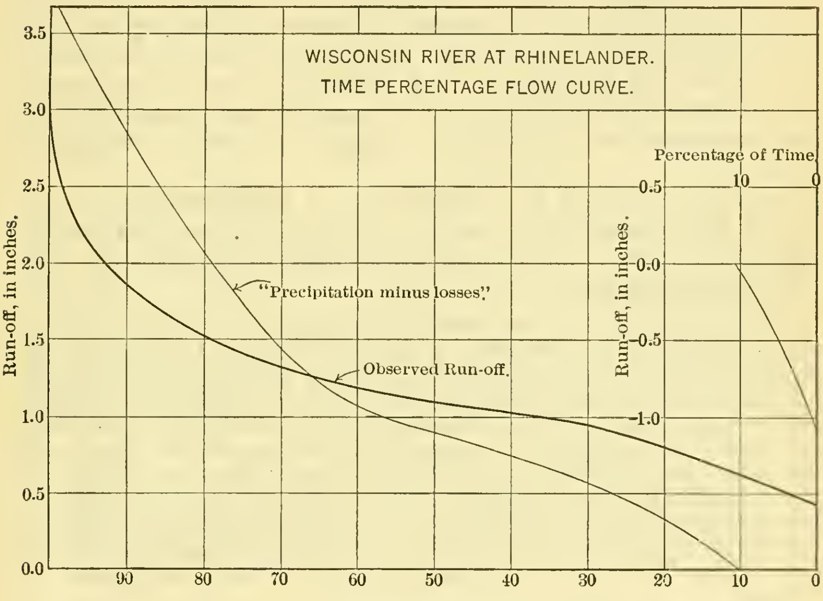


FIG. 40.

losses" would seem to indicate for any given month in a period from September 1st to February 28th or 29th. Apparently, the precipitation during September, October, and part of November, in this section of the country, is used to replenish ground-water lost during the growing period, and the fact that the observed run-off for the period from the middle of November to the latter part of February is less than the run-off computed from "precipitation minus losses," is due entirely to conditions of temperature.

The method which is now followed by the Water Resources Branch of the United States Geological Survey in determining the winter

Mr. Hoyt. flow of streams,* affords a means by which the effect of temperature below freezing can readily be determined for any particular stream.

Table 48 shows the actual run-off, in second-feet per square mile, in 10-day periods, from November 20th, 1914, to March 10th, 1915, for a number of Wisconsin rivers. During a period from the last part of November to the last part of December, the mean temperature dropped from about 30° Fahr. to 0° Fahr. During this same period the mean run-off, in second-feet per square mile, considering all the streams, dropped from 0.56 to about 0.36 sec-ft. per sq. mile, or 36 per cent. The greatest drop for the mean of all the streams was 68% in the Lake Superior Basin. It will be noticed, however, that though most of the streams show a tendency to decrease in flow, the variation in the different basins is startling. Fig. 41 shows the average run-off for the five principal basins in Wisconsin for the period considered, and indicates that the drop in flow was gradual until about February 10th, when the effect of higher temperature is apparent in the stream flow.

In connection with the determination of the distribution of the annual flow throughout the various months of the year, Mr. Meyer (page 617†) states as follows, which has direct reference to the flow during the frozen period of the year:

"It is important to know whether the temperature during the winter suddenly drops to a point where a heavy ice cover is formed over the stream in one or two days, almost shutting off the flow until the stage has sufficiently increased and the slope has been sufficiently equalized and increased, so that the combined area of cross-section and gradient are ample to overcome the increased friction due to ice cover, and thus carry approximately the quantity of water that was flowing in the stream before the freeze-up. It is important to know, further, whether the ground-water table lies so close to the surface that a large portion of the ground-water which would otherwise maintain stream flow during the winter, is frozen up and held until the following spring."

The writer considers it extremely doubtful whether it is possible to determine even approximately the effects of the foregoing conditions without actual measurements. Observation and detailed study at more than seventy-five stations in Minnesota and Wisconsin during the last 3 years have not enabled him to point out any real characteristics as the same year after year. Conditions of temperature differ so radically each winter that there seems to be no method of reasoning by which one can determine in advance, from temperature alone, what the stream flow will be. We know, in a general way, that the flow will drop off materially during cold periods, but there seems to be no relation

* Outlined in Water-Supply Paper 337, "Effects of Ice on Stream Flow."

† *Proceedings*, Am. Soc. C. E., for March, 1915.

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RUN-OFF, IN SECOND-FEET PER SQUARE MILE, FOR THE FIVE MAIN DRAINAGE BASINS IN WISCONSIN, FOR THE PERIOD, NOVEMBER 21st, 1914, MARCH 10th, 1915.

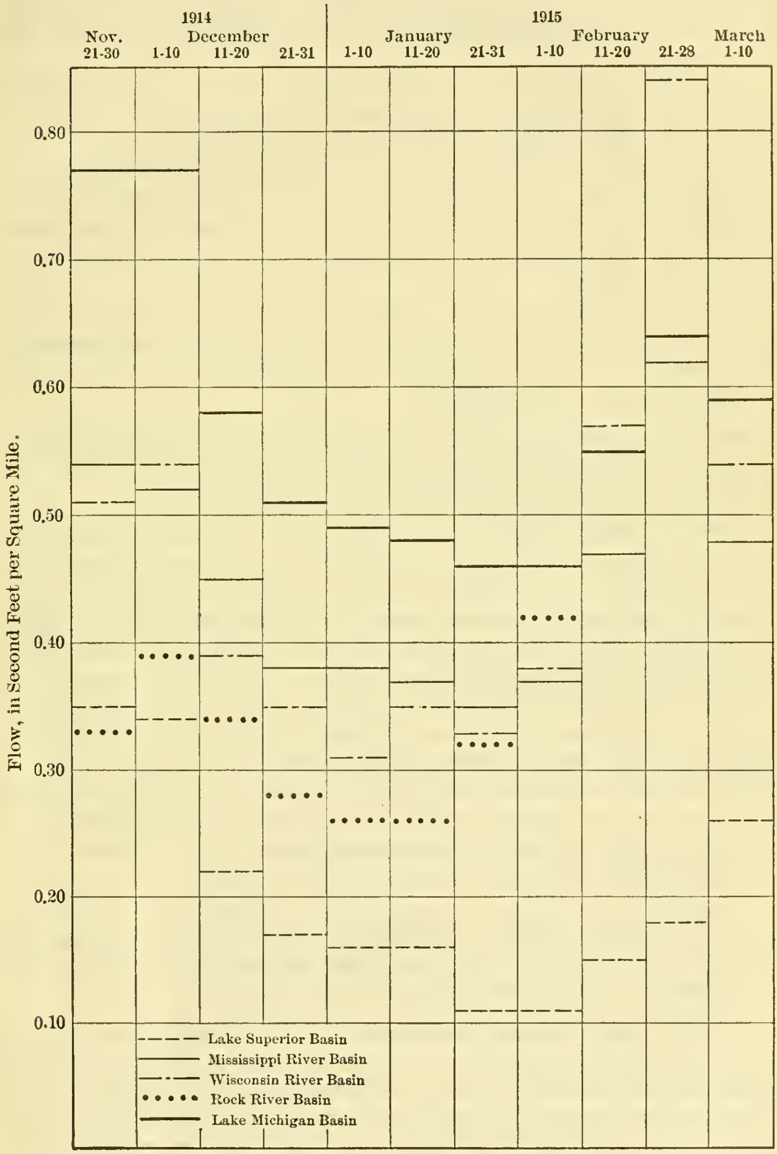

FIG. 41.

Mr. Hoyt

between the drop in the different basins. The Survey is now using a method whereby the stream flow during the winter period can be very closely estimated, but it depends on the use of detailed field observations in connection with current-meter measurements during such periods, and a close study of observed gauge heights and climatologic conditions.

The large variation in the winter flow of streams which drain areas apparently similar in all respects and are subject to the same conditions of temperature is shown by Table 48, and Fig. 41.

Undoubtedly, in a large part of this country, minimum stream flow occurs during the winter, and if the quantity of this flow is determinable by the method devised by Mr. Meyer without the large expenditures now necessary, he will have performed a great service to the engineering world.

The writer agrees with Mr. Meyer's statement on page 614,* that "an identical recurrence of any given combination of meteorological phenomena * * * is extremely improbable," but not necessarily with the statement on page 615:*

"The engineer is usually much more interested in the total run-off from such a water-shed, up to the point of economical utilization, than in the exact distribution of that run-off through the year."

Perhaps, from the point of view of engineers designing water-power work, the total run-off from a drainage basin and the extreme maximum and minimum—which, by the way, cannot be accurately determined by Mr. Meyer's method—are all that would be desired. There are other engineers, however, who, in their hydraulic studies, desire records of the actual daily and monthly flow. The writer does not doubt that, on streams on which there are gauging stations, Mr. Meyer may—by the use of his method and, say, 50 years or more of accurate climatological data—be able to determine a more accurate mean than one based only on the observed run-off records. As far as the writer knows now, however, the monthly mean will be only an approximation. In the introduction of recent water-supply papers of the United States Geological Survey, the following statement is made:

"The tables of monthly discharge are so arranged as to give only a general idea of the flow at the station, and should not be used for other than preliminary estimates. The determinations of the daily discharge allow more detailed studies in the variation in flow."

By Mr. Meyer's method, only a very rough estimate of the monthly distribution of flow is possible, and if the engineers who are collecting these data throughout the country believe that the actual monthly values of flow should be used only for preliminary estimates, how

* *Proceedings*, Am. Soc. C. E., for March, 1915.

much more care should be taken in using estimates that only approximate the monthly flow. In general, however, the writer believes that Mr. Meyer should receive much credit for the careful way in which he has undertaken his study and the method by which he has presented his results. The writer is of the opinion that this is the first time the factors affecting stream flow have been studied and analyzed in such a way as to permit of their being used intelligently as a means of estimating stream flow. Mr. Hoyt.

The writer would sum up his discussion by saying that undoubtedly the long-time mean for any particular drainage basin on which there are at present stream-gauging stations can be accurately determined by Mr. Meyer's method. He believes, however, that the estimates of monthly discharge and probable maximum and minimum flow, computed from "precipitation minus losses" would at best be only roughly approximate, and that, until a large number of comparisons of computed and observed run-off have been made in all parts of the country, the method should not be used to determine the run-off of a stream for which no actual run-off records are available.

Characteristics of Rock River Drainage Basin, Wisconsin and Illinois.—Above Rockton, Ill., Rock River and its tributaries, Pecatonica and Sugar Rivers, occupy a rectangular basin, which is about 115 miles wide and 60 miles long, and thus differs greatly from the ordinary drainage basin, which is generally longer than it is wide.

"The surface is moderately hilly; it varies in elevation from 750 feet where the river enters the State of Illinois, to 1 100 feet on the crests of the Kettle Range. The rise from the interior of the valley is gradual, and usually the hilltops are not more than 100 feet above the intervening valleys * * *. This low, uneven topography has led to the formation of an intricate tributary system, with numerous small spring-fed lakes, * * *."*

The main part of the drainage basin is underlain with Galena and Trenton limestone, although in the western part there is considerable St. Peter sandstone.†

It has been estimated‡ that the surface may be divided as follows: 30% forest, 57% cultivated land, 8% swamps and uncultivated meadows, and 5% water surface.

Good natural storage is manifested by the well-sustained flow during the winter and during dry periods.

Characteristics of Black River Drainage Basin, Wisconsin.—This basin is in Western Wisconsin, and is about 115 miles long and

* U. S. Bureau of Forestry, *Bulletin No. 44*, p. 10.

† Hotchkiss, W. O., and Steidtmann, "Limestone Road Material of Wisconsin." Wisconsin Survey, *Bulletin No. 34*.

‡ Smith, L. S., "Water Powers of Wisconsin." Wisconsin Survey, *Bulletin No. 20*, p. 287.

Mr. Hoyt. 20 miles in average width. Above Neillsville, where the gauging station is situated, the length of the basin is about 55 miles, the maximum width about 15 miles, up stream from Neillsville being less than 10 miles, and the maximum width in the upper basin less than 20 miles. Except for a narrow strip under the immediate valley of the river, the basin is underlain with Potsdam sandstone, the remainder being underlain with the older formation, made up of igneous and metamorphic rock. The area above Neillsville is overlain with glacial drift.

The elevation of the basin at the source of the river is somewhat more than 1 400 ft., and the elevation of the river at Neillsville is approximately 985 ft. above sea level. The valleys above Neillsville are V-shaped and narrow. The water reaches the river quickly following precipitation.

Characteristics of Wisconsin River Basin, Above Rhinelander, Wis.—This is a bell-shaped basin, about 42 miles long and 14 miles wide at the Wisconsin-Michigan boundary, gradually increasing to about 40 miles wide at Rhinelander. The soil north of Rhinelander is mostly sand, intermingled with gravel and glacial drift. Little rock outcrops in the entire basin. About 10% of the land is under cultivation or covered with grass; the remainder is second growth and timber. According to A. A. Babcock, Manager of the Wisconsin Valley Improvement Company, approximately 21.10% of the area above Rhinelander is lakes, 0.13% streams, 18.65% swamp, 4.12% kettle holes, and the remainder is high land.

The percentages were computed by Mr. Babcock from the best maps available, many of which were made under his direction. About 45% of the entire area above the gauging station at Rhinelander is under reservoir control.

"The operation [of these reservoirs] throughout the year is about as follows: In the spring of the year as soon as the natural flow of the river below the reservoirs is sufficient to supply the need of the power plants nearest the headwaters, the gates at the outlets of the lakes are closed and water collected. When the summer drought begins the gates are slightly opened and the stored water used to increase the flow in the river. During the fall of the year the natural flow of the river again increases as a result of the fall rains, and the gates are closed and the reservoirs partially refilled. This stored water, and any remaining from the summer period, is then gradually used during the late fall months and winter, the longer period of drought when the precipitation is slight and being stored in the form of snow."*

Mr. Stewart, in the same report (pages 22 and 23), speaking of the drainage basin, says of that portion which is in Vilas, Oneida, and Lincoln Counties:

* Stewart, C. B., "Storage Reservoirs at the Headwaters of the Wisconsin River and Their Relation to Stream Flow," 1911, p. 10.

"The soil of these counties consists of glacial drift of porous sandy material, varying from pure sand in some places to sand mixed with a slight amount of clay in others. * * * The land is more or less rolling but the slopes are gradual with the variations in elevation not exceeding about 100 feet. The most characteristic feature of the topography, especially in Vilas and Oneida Counties, is the sand hills with rounded tops, interspersed with circular or elongated valleys, and occupied by lakes with or without outlets * * *. The character of the soil and topography, together with absence of any erosion indicates that surface flow from rains will not occur except during periods of the early spring when the ground is frozen." Mr. Hoyt.

Characteristics of Wisconsin River Basin, Above Merrill, Wis.—This basin is about 80 miles long and 45 miles wide at its widest part. Conditions as regards geology and soil are much the same as those mentioned for Wisconsin River between Rhinelander and Merrill, Wis. About 580 of the 2 630 sq. miles above Merrill are under control by storage reservoirs, there being about seventeen reservoirs operated by the Wisconsin Valley Improvement Company, with a total capacity of more than 4 000 000 000 cu. ft. Details as to the operation of these reservoirs are given under the heading "Characteristics of Wisconsin River Basin, Above Rhinelander, Wis." According to Mr. A. A. Babcock, this area is made up as follows: 6.39% lakes, 0.30% streams, 21.25% swamps, 3.06% kettle holes, and the remainder high land.

Characteristics of Wisconsin River Basin Between Rhinelander and Merrill, Wis., Including Tomahawk River.—Between Rhinelander and Merrill, Wisconsin River drains an area of triangular shape, about 60 miles long and 45 miles wide at the mouth of Tomahawk River. It is underlain with Pre-Cambrian rock and so deeply covered with drift that rocks outcrop in few places. Of the 1 520 sq. miles which make up this area, about one-third is drained by Tomahawk River and its tributaries. Of this latter area, according to Mr. Babcock, approximately 8.62% is made up of lakes, 0.32% of streams, 24.45% of swamps, 3.61% of kettle holes, and the remainder is high land. Of the total area between Rhinelander and Merrill, from 4 to 6% is under reservoir control. These reservoirs are operated in the same manner as those on Wisconsin River above Rhinelander.

In computing Tables 38 to 48, inclusive, it should be remembered that the writer had to work with the small-scale curves of Figs. 17 and 19, and that undoubtedly there are errors, due to this fact. It is believed that in the final discussion Mr. Meyer should include these two curves on a scale at least twice as large. The writer also used the curves without taking into account abnormal precipitations or low storage conditions, which may change the values somewhat. No attempt has been made to correct final results for changes in ground storage or for seepage flow.

Mr. Hoyt.

TABLE 38.—ROCK RIVER BELOW MOUTH OF PECATONICA RIVER, AT ROCKTON, ILL.

Drainage area, 6 290 sq. miles.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	(a) Monthly precipitation, in inches.	LOSS FROM LAND AREA.				Total loss, in inches.	Precipitation minus total loss, in inches.	(b) Observed run-off, in inches.
			Transpiration.		Evaporation.				
			From curve.	Actual.	From curve.	Actual.			
1903									
November......	35	1.18	....	....	0.30	0.31	0.31	0.87	0.50
December......	17	1.44	....	....	0.40	0.42	0.42	1.02	0.39
1904									
January........	13	0.87	....	....	0.25	0.26	0.26	0.61	0.29
February.......	12	1.24	....	....	0.30	0.31	0.31	0.93	0.28
March..........	33	3.62	....	....	1.05	1.10	1.10	2.52	2.62
April...........	42	2.58	0.21	0.16	1.00	1.05	1.26	1.32	1.92
May.............	59	3.63	1.70	1.27	1.75	1.84	3.11	0.52	0.99
June...........	68	1.89	2.30	1.72	1.22	1.28	3.00	—1.11	0.45
July............	71	3.39	2.50	1.87	2.00	2.10	3.97	—0.58	0.30
August.........	69	4.34	2.20	1.65	2.15	2.26	3.91	0.43	0.28
September......	63	5.75	1.70	1.27	2.45	2.57	3.84	1.91	0.43
October.........	53	2.27	0.88	0.66	1.00	1.05	1.66	0.61	0.57
	44.6	32.20	11.49	8.60	13.87	14.55	23.15	9.05	9.02
1904									
November......	42	0.27	....	....	0.20	0.21	0.21	0.06	0.34
December......	23	2.56	....	....	0.45	0.47	0.47	2.09	0.34
1905									
January........	12	1.12	....	....	0.30	0.31	0.31	0.81	0.40
February.......	13	1.54	....	....	0.30	0.31	0.31	1.23	0.39
March..........	36	2.90	....	....	0.97	1.03	1.03	1.87	2.33
April...........	47	2.52	0.75	0.56	1.10	1.16	1.72	0.80	1.80
May.............	57	6.22	1.52	1.14	2.42	2.54	3.68	2.54	1.23
June............	68	4.12	2.30	1.73	2.20	2.31	4.04	0.08	1.15
July............	71	3.52	2.50	1.87	2.00	2.10	3.97	—0.45	0.73
August..........	73	4.25	2.52	1.89	2.30	2.42	4.31	—0.06	0.48
September......	67	1.60	2.05	1.54	1.00	1.05	2.59	—0.99	0.48
October.........	51	3.46	0.70	0.52	1.30	1.37	1.89	1.57	0.47
	46.6	34.08	12.31	9.25	14.54	15.28	24.53	9.55	10.05

(a) Taken from mean of nine U. S. Weather Bureau precipitation stations in or adjacent to Rock River Drainage Basin above Rockton, Ill.

(b) Water Resources of Illinois, Report of Rivers and Lakes Commission of Illinois, by U. S. Geological Survey, 1914.

TABLE 38.—(*Continued.*)

Mr. Hoyt.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area. Transpiration. From curve.	Loss from Land Area. Transpiration. Actual.	Loss from Land Area. Evaporation. From curve.	Loss from Land Area. Evaporation. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
1905									
November......	38	2.42	....	....	0.60	0.63	0.63	1.79	0.45
December......	28	1.44	....	....	0.25	0.26	0.26	1.18	0.51
1906									
January........	26	2.90	....	....	0.48	0.50	0.50	2.40	1.76
February.......	22	1.52	....	....	0.50	0.52	0.52	1.00	1.64
March..........	27	2.34	....	....	0.72	0.76	0.76	1.58	2.29
April...........	51	1.49	1.10	0.83	0.80	0.84	1.67	—0.18	1.63
May............	58	3.14	1.60	1.20	1.60	1.68	2.88	0.26	0.67
June............	67	4.05	2.22	1.66	2.15	2.26	3.92	0.13	0.42
July............	72	2.11	2.60	1.95	1.40	1.47	3.42	—1.31	0.28
August.........	74	6.36	2.61	1.95	3.15	3.31	5.26	1.10	(a) 0.35
September......	69	3.21	2.20	1.64	1.80	1.81	3.45	—0.24	(a) 0.20
October........	52	2.55	0.78	0.58	1.00	1.05	1.63	0.92	(a) 0.30
	48.6	33.53	13.11	9.81	13.91	15.09	24.90	8.63	10.50
1906									
November......	38	2.76	....	....	0.71	0.75	0.75	2.01	0.47
December......	27	1.52	....	....	0.25	0.26	0.26	1.26	0.61
1907									
January........	19	2.94	....	....	0.58	0.61	0.61	2.33	1.15
February.......	23	0.43	....	....	0.50	0.52	0.52	—0.09	(a) 0.80
March..........	40	2.16	....	....	0.90	0.95	0.95	1.21	0.88
April......	40	3.22	....	....	1.15	1.21	1.21	2.01	1.32
May............	51	3.17	1.1	0.82	1.40	1.47	2.29	0.88	0.74
June............	67	4.80	2.2	1.65	2.40	2.52	4.17	0.63	0.93
July............	73	6.59	2.6	1.95	3.25	3.41	5.36	1.23	1.09
August.........	69	3.92	2.2	1.65	2.00	2.10	3.75	0.17	0.73
September......	62	6.05	1.6	1.20	2.52	2.64	3.84	2.21	0.80
October........	49	1.09	0.5	0.38	0.42	0.44	0.82	0.27	0.82
	46 5	38.65	10.2	7.65	16.08	16.88	24.53	14.12	10.34
1907									
November......	38	1.19	....	....	0.35	0.37	0.37	0.82	0.82
December......	29	1.55	....	....	0.25	0.26	0.26	1.29	0.36
1908									
January........	26	1.21	....	....	0.62	0.65	0.65	0.56	0.43
February.......	25	1.85	....	....	0.66	0.69	0.69	1.16	1.13
March..........	38	2.73	....	....	0.97	1.02	1.02	1.71	1.99
April...........	52	4.17	1.18	0.89	1.75	1.84	2.73	1.44	1.19
May............	61	5.92	1.80	1.35	2.50	2.62	3.97	1.95	1.67
June............	67	3.48	2.22	1.66	1.90	2.00	3.66	—0.18	1.14
July............	73	2.59	2.68	2.01	1.65	1.73	3.74	—1.15	0.65
August.........	70	3.52	2.30	1.72	1.89	1.97	3.69	—0.17	0.33
September.....	68	1.51	2.12	1.59	0.95	1.00	2.59	—1.08	0.26
October........	52	1.24	0.78	0.59	0.55	0.58	1.17	0.07	0.28
	49.9	30.96	13.08	9.81	14.04	14.73	24.54	6.42	10.25

(a) Estimated.

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TABLE 39.—SUMMARY OF DATA AND COMPUTATIONS FOR ROCK RIVER DRAINAGE BASIN ABOVE ROCKTON, ILL.

Drainage area, 6 290 sq. miles.

Year.	(a) Rainfall.	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	(a) Observed run-off.
1904	32.20	14.55	8.60	23.15	9.05	9.02
1905	34.08	15.28	9.25	24.53	9.55	10.05
1906	33.53	15.09	9.81	24.90	8.63	(b) 10.50
1907	38.65	16.88	7.65	24.53	14.12	10.34
1908	30.96	14.73	9.81	24.54	6.42	10.25
Total....	169.42	76.53	45.12	121.65	47.77	50.16
Mean...	33.88	15.31	9.02	24.33	9.55	10.03

(a) November 1st of previous year to October 31st of given year.

(b) Run-off estimated from August 1st to October 31st.

Mr. Hoyt.

TABLE 40.—DATA AND COMPUTATIONS FOR BLACK RIVER AT NEILLSVILLE, WIS.

Drainage area, 774 sq. miles.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	(a) Monthly precipitation, in inches.	Loss from Land Area. Transpiration. From curve.	Transpiration. Actual.	Evaporation. From curve.	Evaporation. Actual.	Total loss, in inches.	Precipitation, minus total loss, in inches.	(b) Observed run-off, in inches.
1905									
November	32	1.25	....	....	0.22	0.19	0.19	1.06	0.56
December	22	0.80	....	....	0.20	0.18	0.18	0.62	0.43
1906									
January	20	2.01	....	....	0.60	0.53	0.53	1.48	(c) 0.20
February	16	0.54	....	....	0.40	0.35	0.35	0.19	(c) 0.20
March	22	1.91	....	....	0.68	0.60	0.60	1.31	(c) 0.50
April	47	1.39	0.72	0.50	0.62	0.55	1.05	0.34	5.57
May	55	5.82	1.40	0.98	2.28	2.00	2.98	2.84	2.16
June	63	4.23	1.95	1.36	2.10	1.85	3.21	1.02	1.05
July	68	2.85	2.30	1.61	1.70	1.49	3.10	−0.25	0.27
August	69	3.40	2.20	1.54	1.82	1.60	3.14	0.26	0.28
September	61	3.40	1.80	1.26	1.65	1.45	2.71	0.69	0.40
October	47	3.02	0.35	0.24	1.05	0.92	1.16	1.86	0.44
	43.7	30.62	10.72	7.49	13.32	11.71	19.20	11.42	12.06
1906									
November	32	2.70	....	....	0.52	0.46	0.46	2.24	1.06
December	21	1.24	....	....	0.32	0.28	0.28	0.96	0.80
1907									
January	13	1.57	....	...	0.28	0.25	0.25	1.32	(c) 0.20
February	17	0.65	....	....	0.44	0.39	0.39	0.26	(c) 0.20
March	35	1.82	....	....	0.78	0.69	0.69	1.13	3.11
April	36	1.34	....	....	0.68	0.60	0.60	0.74	1.84
May	47	2.89	0.72	0.50	1.20	1.05	1.55	1.34	1.05
June	63	3.61	1.95	1.36	1.88	1.65	3.01	0.60	0.26
July	69	2.69	2.20	1.54	1.55	1.36	2.90	−0.21	0.73
August	64	4.73	1.80	1.26	2.12	1.86	3.12	1.61	0.14
September	57	4.16	1.20	0.84	1.72	1.51	2.35	1.81	0.68
October	44	1.02	0.10	0.07	0.40	0.35	0.42	0.60	0.15
	41.5	28.42	7.97	5.57	11.89	10.45	16.02	12.40	10.22

(a) Mean of five U. S. Weather Bureau precipitation stations in or adjacent to Black River Drainage Basin above Neillsville.

(b) Run-off by U. S. Geological Survey. Figures revised on account of error of drainage basin as published in Water Supply Papers 171, 207, 245, and 265.

(c) Estimated.

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Hoyt.

TABLE 40.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.				Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
			Transpiration.		Evaporation.				
			From curve.	Actual.	From curve.	Actual			
1907									
November......	32	0.94	....	....	0.20	0.18	0.18	0.76	0.11
December......	23	0.57	....	....	0.20	0.18	0.18	0.39	0.08
1908									
January........	18	0.65	....	....	0.42	0.37	0.37	0.28	0.10
February.......	17	1.37	....	....	0.45	0.40	0.40	0.97	0.10
March..........	32	2.31	....	....	0.82	0.72	0.72	1.59	1.00
April............	44	3.29	0.42	0.29	1.25	1.10	1.39	1.90	3.31
May.............	55	5.21	1.40	0.97	2.10	1.85	2.82	2.39	2.92
June............	63	6.10	1.95	1.36	2.65	2.33	3.69	2.41	2.03
July.............	69	3.92	2.36	1.65	2.15	1.89	3.54	0.38	1.26
August..........	66	1.92	1.98	1.39	1.10	0.96	2.35	−0.43	0.08
September......	64	2.98	1.80	1.26	1.50	1.32	2.58	0.40	0.05
October.........	48	2.09	0.42	0.29	0.80	0.70	0.99	1.10	0.17
	44.2	31.35	10.33	7.21	13.64	12.00	19.21	12.14	11.21
1908									
November......	34	1.50	....	....	0.32	0.28	0.28	1.22	0.26
December......	19	1.28	....	....	0.40	0.35	0.35	0.93	0.17
1909									
January........	16	0.77	....	....	0.32	0.28	0.28	0.49	0.15
February.......	20	1.95	....	....	0.56	0.49	0.49	1.46	0.07
March..........	27	1.40	....	....	0.62	0.55	0.55	0.85	0.21
April............	38	3.40	....	....	1.12	0.98	0.98	2.42	No
May.............	53	2.92	1.25	0.88	1.35	1.19	2.07	0.85	run-off
June............	65	4.22	2.08	1.45	2.15	1.89	3.34	0.88	data,
July.............	68	2.55	2.30	1.61	1.52	1.34	2.95	−0.40	Apr.,
August..........	70	2.88	2.30	1.61	1.67	1.47	3.08	−0.20	1909,
September.....	58	4.46	1.29	0.90	1.85	1.63	2.53	1.93	to
October........	44	2.09	0.10	0.07	0.66	0.58	0.65	1.45	Nov.,
									1913.
	42.6	29.42	9.32	6.52	12.54	11.03	17.55	11.87	

Mr. Hoyt.

TABLE 40.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.				Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
			Transpiration.		Evaporation.				
			From curve.	Actual.	From curve.	Actual.			
1909									
November......	39	5.10	....	....	1.21	1.06	1.06	4.04	....
December......	11	1.41	....	....	0.21	0.18	0.18	1.23	....
1910									
January........	13	1.44	....	....	0.32	0.28	0.28	1.16	....
February.......	12	0.30	....	....	0.28	0.25	0.25	0.05	....
March..........	43	0.04	....	....	0.31	0.27	0.27	−0.23	....
April...........	47	2.84	0.75	0.52	1.17	1.03	1.55	1.29	....
May	51	3.05	1.08	0.76	1.40	1.23	1.99	1.06	....
June...........	67	0.59	1.22	0.50	0.50	0.44	1.29	−0.70	....
July	72	2.61	2.60	1.82	1.65	1.45	3.27	—0.66	....
August.........	68	4.34	2.12	1.48	2.10	1.85	3.33	1.01	....
September......	58	2.35	1.29	0.90	1.08	0.95	1.85	0.50	....
October	51	1.60	0.69	0.48	0.68	0.60	1.08	0.52	....
	44.3	25.67	9.75	6.81	10.91	9.59	16.40	9.27	....
1910									
November......	26	0.69	....	....	0.15	0.13	0.13	0.56	....
December.......	17	0.77	....	....	0.20	0.17	0.17	0.60	....
1911									
January........	15	1.02	....	....	0.37	0.32	0.32	0.70	....
February	22	1.24	....	....	0.48	0.42	0.42	0.82	....
March	33	1.36	....	....	0.70	0.62	0.62	0.74	....
April	42	1.11	0.22	0.15	0.62	0.55	0.70	0.41	....
May	61	5.82	1.82	1.27	2.50	2.20	3.47	2.35	....
June............	69	5.12	2.38	1.66	2.60	2.28	3.94	1.18	....
July	71	4.94	2.50	1.75	2.60	2.28	4.03	0.91	...
August.........	66	2.94	1.97	1.37	1.52	1.34	2.71	0.23	....
September......	59	6.92	1.38	0.97	2.60	2.28	3.25	3.67	....
October.........	45	9.37	0.18	0.12	2.25	1.98	2.10	7.27	....
	43.7	41.30	10.51	7.29	16.59	14.57	21.86	19.44	....

Mr. Hoyt.

TABLE 40.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area. Transpiration. From curve.	Transpiration. Actual.	Evaporation. From curve.	Evaporation. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
1911									
November	24	1.95	....	....	0.36	0.32	0.32	1.63	....
December	24	2.52	....	....	0.45	0.40	0.40	2.12	....
1912									....
January	— 6	0.57	....	....	0.05	0.04	0.04	0.53	....
February	11	0.13	....	....	0.25	0.22	0.22	—0.09	....
March	22	0.50	....	....	0.48	0.42	0.42	0.08	....
April	47	3.14	0.72	0.51	1.30	1.14	1.65	1.49	....
May	57	5.85	1.52	1.06	2.32	2.04	3.10	2.75	....
June	65	1.04	2.09	1.46	0.82	0.72	2.18	—1.14	....
July	69	6.76	2.36	1.65	2.80	2.46	4.11	2.65	....
August	65	8.08	1.87	1.31	3.25	2.85	4.16	3.92	....
September	61	4.01	1.53	1.07	1.80	1.57	2.64	1.37	....
October	49	1.80	0.52	0.36	0.70	0.62	0.98	0.82	....
	40.6	36.35	10.12	7.42	14.58	12.80	20.22	16.13	....
1912									
November	35	0.83	....	....	0.20	0.18	0.18	0.65	....
December	23	2.27	....	....	0.42	0.37	0.37	1.90	....
1913									....
January	15	0.58	....	....	0.31	0.27	0.27	0.31	....
February	10	0.93	....	....	0.20	0.18	0.18	0.75	...
March	25	3.18	....	....	0.80	0.70	0.70	2.48	....
April	47	2.39	0.74	0.52	1.00	0.88	1.40	0.99	....
May	55	5.63	1.40	0.97	2.20	1.94	2.91	2.72	....
June	70	2.47	2.42	1.69	1.52	1.34	3.03	—0.56	....
July	69	6.04	2.36	1.65	2.95	2.60	4.25	1.79	....
August	70	2.26	2.30	1.61	1.32	1.16	2.77	—0.51	....
September	61	3.19	1.54	1.08	1.50	1.32	2.40	0.79	....
October	38	1.94	....	....	0.46	0.40	0.40	1.54	....
	43.2	31.71	10.76	7.52	12.88	11.34	18.86	12.85	....
1913									
November	38	1.51	....	....	0.40	0.35	0.35	1.18	(c) 0.30
December	28	1.12	....	....	0.12	0.06	0.06	0.06	(c) 0.10
1914									
January	22	1.95	....	....	0.60	0.53	0.53	1.42	0.18
February	7	0.51	....	....	0.14	0.12	0.12	0.39	0.25
March	29	1.59	....	....	0.70	0.62	0.62	0.97	1.33
April	43	3.08	0.32	0.22	1.20	1.06	1.28	1.80	2.87
May	60	4.48	1.76	1.23	2.08	1.83	3.06	1.42	1.94
June	66	9.21	2.15	1.51	4.05	3.56	5.07	4.14	4.11
July	75	2.40	2.92	2.04	1.60	1.41	3.45	—1.05	0.76
August	69	4.48	2.20	1.55	2.23	1.96	3.51	0.97	0.23
September	60	4.61	1.45	1.02	1.90	1.67	2.69	1.92	1.29
October	53	2.06	0.85	0.59	1.92	1.68	2.27	—0.21	0.69
	45.7	36.00	11.65	8.15	16.95	14.85	23.01	12.99	14.15

(c) Partly estimated.

Mr. Hoyt.

TABLE 41.—SUMMARY OF DATA AND COMPUTATIONS FOR BLACK RIVER BASIN, ABOVE NEILLSVILLE, WIS.

Drainage area, 774 sq. miles.

Year.	(a) Rainfall.	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	(b) Observed run-off.
1906	30.62	11.71	7.49	19.20	11.42	12.93
1907	28.42	10.45	5.57	16.02	12.40	8.35
1908	31.35	12.00	7.21	19.21	12.14	11.49
1909	29.42	11.03	6.52	17.55	11.87	......
1910	25.67	9.59	6.81	16.40	9.27	......
1911	41.30	14.57	7.29	21.86	19.44	......
1912	(c)36.35	12.80	7.42	20.22	16.13	......
1913	31.71	11.34	7.52	18.86	12.85	......
1914	36.00	14.85	8.15	23.07	12.99	13.80
Total...	290 84	108.34	64.68	173.09	118.51	(d) 46.57
Mean...	32.32	12.04	7.19	19.23	13.18	(d) 11.64

(a) November 1st of previous year to October 31st of given year.

(b) Run-off, from March 1st of given year to February 28th or 29th of following year.

(c) 35.11 in. from May to October.

(d) For partial period.

Mr. Hoyt.

TABLE 42.—DATA AND COMPUTATIONS FOR WISCONSIN RIVER BASIN, ABOVE RHINELANDER, WIS.

Drainage area, 1 110 sq. miles.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	(a) Monthly precipitation, in inches.	EVAPORATION FROM WATER AREAS.		LOSS FROM LAND AREAS.				Total loss, in inches.	Precipitation minus total loss, in inches.	(b) Observed run-off, in inches.
					Transpiration		Evaporation.				
			From curve.	Actual.	From curve.	Actual.	From curve.	Actual.			
1908											
November..	32.6	1.37	1.16	0.12	....	....	0.25	0.13	0.25	1.12	0.42
December..	17.4	0.88	0.40	0.04	....	....	0.20	0.11	0.15	0.73	0.78
1909											
January....	15.4	0.52	0.32	0.03	....	....	0.30	0.16	0.19	0.23	0.91
February ..	17.6	1.15	0.51	0.05	....	....	0.40	0.21	0.26	0.89	0.77
March......	25.0	1.09	1.00	0.10	....	....	0.60	0.32	0.42	0.67	0.67
April.......	34.0	3.33	1.80	0.18	....	....	1.00	0.53	0.71	2.62	1.04
May........	51.7	1.90	2.32	0.23	1.15	0.71	0.95	0.50	1.44	0.46	1.94
June.......	65.2	3.10	3.68	0.37	2.10	1.30	1.72	0.91	2.58	0.52	1.26
July........	66.8	6.56	4.08	0.41	2.22	1.38	3.10	1.64	3.43	3.13	1.29
August.....	67.8	2.93	4.18	0.42	2.30	1.43	1.58	0.84	2.69	0.24	1.71
September..	57.4	2.01	3.15	0.32	1.20	0.75	0.98	0.52	1.59	0.42	1.09
October	43.1	1.22	1.92	0.19	....	....	0.40	0.21	0.40	0.82	0.83
	41.1	26.06	24.52	2.46	8.97	5.57	11.48	6.08	14.11	11.95	12.71
1909											
November..	37.0	5.10	1.48	0.15	....	....	1.12	0.59	0.74	4.36	1.53
December..	16.4	0.99	0.40	0.04	....	....	0.28	0.15	0.19	0.80	1.11
1910											
January....	13.8	0.81	0.28	0.03	....	....	0.30	0.16	0.19	0.62	1.21
February ..	11.4	1.20	0.28	0.03	....	....	0.25	0.13	0.16	1.04	1.23
March	40.0	0.38	2.50	0.25	....	....	0.42	0.22	0.47	−0.09	1.18
April.......	46.8	2.01	1.92	0.19	0.72	0.45	0.92	0.49	1.13	0.88	1.19
May........	49.2	2.54	2.11	0.21	0.92	0.57	1.18	0.63	1.41	1.13	1.03
June	65.7	0.34	3.72	0.37	1.15	0.71	0.40	0.21	1.29	−0.95	0.60
July........	69.2	1.91	4.16	0.42	2.35	1.45	1.28	0.68	2.55	−0.64	0.56
August.....	66.2	3.14	4.05	0.40	1.95	1.21	1.68	0.89	2.50	0.64	0.53
September..	52.6	2.32	3.02	0.30	0.88	0.55	0.92	0.49	1.34	0.98	0.60
October	48.6	1.95	2.38	0.24	0.52	0.32	0.60	0.32	0.88	1.07	0.56
	43.2	22.69	26.30	2.63	8.49	5.26	9.38	4.96	12.85	9.84	11.33

(a) Mean determined from seven U. S. Weather Bureau stations, in and adjacent to the Wisconsin Basin above Rhinelander.

(b) As determined by the U. S. Geological Survey.

Mr. Hoyt.

TABLE 42.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Evaporation from Water Areas.		Loss from Land Areas.				Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
					Transpiration.		Evaporation.				
			From curve.	Actual.	From curve.	Actual.	From curve.	Actual.			
1910											
November..	25.7	1.05	0.78	0.08	....	....	0.20	0.11	0.19	0.86	0.60
December..	15.9	0.71	0.35	0.04	....	....	0.30	0.16	0.20	0.51	0.56
1911											
January ...	12.4	0.73	0.28	0.03	....	....	0.30	0.16	0.19	0.54	0.84
February ..	20.4	1.08	0.68	0.07	....	....	0.52	0.28	0.35	0.73	0.75
March......	30.2	1.65	1.42	0.14	....	....	0.60	0.32	0.46	1.19	0.97
April.......	42.0	0.64	1.60	0.16	0.22	0.14	0.50	0.26	0.56	0.08	1.12
May	59.4	5.10	3.05	0.30	1.68	1.04	2.20	1.17	2.51	2.59	1.03
June	69.6	1.32	4.18	0.42	2.42	1.50	1.00	0.53	2.45	—1.13	0.72
July	67.8	7.43	3.96	0.40	2.30	1.42	3.95	2.10	3.92	3.51	1.16
August.....	63.8	5.22	3.79	0.38	1.80	1.12	2.32	1.23	2.73	2.49	1.84
September.	56.6	3.04	3.07	0.31	1.20	0.75	1.40	0.74	1.80	1.24	1.45
October	43.4	6.25	1.95	0.20	....	....	1.70	0.90	1.10	5.15	2.79
	42.3	34.22	25.11	2.53	9.62	5.97	14.99	7.96	16.46	17.76	13.83
1911											
November..	24.4	2.82	0.68	0.07	....	...	0.48	0.25	0.32	2.50	1.02
December..	22.9	2.57	1.90	0.19	....	....	0.30	0.16	0.35	2.22	1.43
1912											
January....	—5.4	0.63	0.01	0.00	....	....	0.10	0.05	0.05	0.58	1.04
February ..	11.4	0.21	0.28	0.03	....	....	0.25	0.13	0.16	0.05	0.97
March......	19.8	0.42	0.64	0.06	....	....	0.48	0.25	0.31	0.11	1.04
April.......	42.1	2.76	1.62	0.16	0.22	0.14	1.00	0.53	0.83	1.93	2.21
May	55.2	4.31	2.55	0.26	1.40	0.87	1.85	0.98	2.11	2.20	1.70
June	61.6	2.16	3.29	0.33	1.88	1.17	1.30	0.69	2.19	—0.03	1.33
July	67.1	3.68	3.92	0.40	2.22	1.37	2.00	1.06	2.83	0.85	1.37
August.....	61.8	7.68	3.60	0.37	1.62	1.01	2.95	1.56	2.94	4.74	3.07
September.	58.4	2.76	3.22	0.32	1.28	0.79	1.28	0.68	1.79	0.97	2.43
October	48.0	1.92	2.32	0.23	0.44	0.27	0.70	0.37	0.87	1.05	1.75
	38.9	31.92	24.03	2.42	9.06	5.62	12.69	6.71	14.75	17.12	19.36

Mr. Hoyt.

TABLE 42.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Evaporation from Water Areas.		Loss from Land Areas.				Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
					Transpiration.		Evaporation.				
			From curve.	Actual.	From curve.	Actual.	From curve.	Actual.			
1912											
November..	33.1	0.49	1.21	0.12	....	....	0.15	0.08	0.20	0.29	1.76
December..	21.6	2.07	0.56	0.06	....	....	0.42	0.22	0.28	1.79	1.53
1913											
January....	14.0	0.53	0.30	0.03	....	....	0.25	0.13	0.16	0.37	0.77
February ..	8.4	0.96	0.20	0.02	....	....	0.18	0.10	0.12	0.84	0.70
March	21.6	2.47	0.72	0.07	....	....	0.65	0.34	0.41	2.06	1.18
April.......	43.8	1.70	1.80	0.18	0.42	0.26	0.80	0.42	0.86	0.84	2.11
May........	52.0	3.68	2.35	0.24	1.08	0.67	1.60	0.85	1.76	1.92	1.65
June.	65.8	3.35	3.78	0.38	2.15	1.33	1.88	1.00	2.71	0.64	1.27
July........	65.6	5.92	3.76	0.38	2.15	1.33	2.70	1.43	3.14	2.78	1.37
August.....	66.4	3.14	4.02	0.40	1.97	1.22	1.70	0.90	2.62	0.52	1.04
September..	56.9	3.81	3.12	0.31	1.20	0.75	1.58	0.84	1.90	1.91	1.18
October....	44.7	3.31	2.02	0.20	0.10	0.06	1.08	0.57	0.83	2.48	1.74
	41.1	31.43	23.84	2.39	9.07	5.62	12.69	6.88	14.99	16.64	16.30
1913											
November..	36.8	1.51	1.45	0.14	....	....	0.40	0.21	0.35	1.16	1.58
December..	28.3	0.15	0.90	0.09	....	....	0.22	0.12	0.21	−0.06	1.27
1914											
January....	19.8	1.44	0.48	0.05	....	....	0.35	0.19	0.24	1.20	1.05
February ..	6.6	0.40	0.16	0.02	....	....	0.18	0.10	0.12	0.28	0.95
March......	24.5	1.66	0.95	0.09	....	....	0.62	0.33	0.42	1.24	1.08
April.......	38.5	3.44	1.40	0.14	....	....	1.15	0.61	0.75	2.69	1.25
May........	56.5	2.18	3.08	0.31	1.48	0.92	1.15	0.61	1.84	0.34	1.27
June........	61.8	5.62	3.34	0.33	1.88	1.16	2.45	1.30	2.79	2.83	1.15
July........	71.1	5.09	4.45	0.44	2.50	1.55	2.72	1.44	3.43	1.66	1.96
August.....	64.6	6.11	3.82	0.38	1.89	1.17	2.60	1.38	2.93	3.18	1.89
September.	58.2	2.62	3.25	0.32	1.30	0.81	1.20	0.64	1.77	0.85	1.47
October....	52.8	0.98	2.78	0.28	0.85	0.53	0.48	0.25	1.06	−0.08	1.02
	43.2	31.20	26.06	2.59	9.90	6.14	13.52	7.18	15.91	15.29	16.38

Mr. Hoyt.

TABLE 43.—Summary of Data and Computations for Wisconsin River Basin, Above Rhinelander, Wis.

Drainage area, 1 110 sq. miles.

Year.	(a) Rainfall.	Evaporation.		Transpiration.	Total loss.	Precipitation minus total loss.	(b) Observed run-off.
		Water.	Land.				
1909	26.06	2.46	6.08	5.57	14.11	11.95	15.42
1910	22.69	2.63	4.96	5.26	12.85	9.84	8.79
1911	34.22	2.53	7.96	5.97	16.46	17.76	15.61
1912	31.92	2.42	6.71	5.62	14.75	17.12	19.80
1913	31.43	2.39	6.88	5.62	14.99	16.64	16.29
1914	31.20	2.59	7.18	6.14	15.91	15.29	14.69
Total... ...	177.52	15.02	39.77	34.18	89.07	88.60	90.60
Average....	29.60	2.50	6.63	5.70	14.83	14.77	15.10

(a) Precipitation, November 1st of preceding year to October 31st of given year.
(b) Run-off, March 1st of given year to February 28th or 29th of following year.

Mr. Hoyt.

TABLE 44.—DATA AND COMPUTATIONS FOR WISCONSIN RIVER BASIN ABOVE MERRILL, WIS.

Drainage area, 2 630 sq. miles.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	(a) Monthly precipitation, in inches.	Loss from Land Area. Transpiration. From curve.	Transpiration. Actual.	Evaporation. From curve.	Evaporation. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	(b) Observed run-off, in inches.
1908									
November.....	32.6	1.47	....	....	0.25	0.17	0.17	1.30	0.57
December.....	17.4	1.09	....	....	0.20	0.13	0.13	0.96	0.65
1909									
January.......	15.4	0.53	....	....	0.42	0.28	0.28	0.25	0.65
February.. ..	17.6	1.26	....	....	0.50	0.33	0.33	0.93	0.62
March.........	25.0	1.29	....	....	0.62	0.41	0.41	0.88	0.57
April	34.0	2.95	....	....	0.92	0.61	0.61	2.34	1.92
May	51.7	2.11	1.15	0.76	1.00	0.66	1.43	0.68	3.14
June...........	65.2	3.36	2.10	1.39	1.90	1.25	2.64	0.72	1.50
July...........	66.8	5.26	2.22	1.47	2.60	1.72	3.19	2.07	1.06
August........	67.8	2.41	2.30	1.52	1.45	0.96	2.48	—0.07	0.81
September	57.4	2.45	1.20	0.79	1.15	0.76	1.55	0.90	0.70
October	43.1	1.71	....	....	0.52	0.34	0.34	1.37	0.62
	41.1	25.89	8.97	5.93	11.53	7.62	13.55	12.34	12.81
1909									
November.....	37.0	4.48	....	....	1.05	0.69	0.69	3.79	1.80
December......	16.4	0.93	....	....	0.35	0.23	0.23	0.70	1.04
1910									
January.......	13.8	0.76	....	....	0.38	0.25	0.25	0.51	0.92
February......	11.4	0.97	....	....	0.25	0.17	0.17	0.80	0.82
March..........	40.0	0.29	....	....	0.42	0.28	0.28	0.01	1.29
April	46.8	2.46	0.72	0.48	1.10	0.73	1.21	1.25	1.75
May......... ...	49.2	2.42	0.92	0.61	1.15	0.76	1.37	1.05	1.21
June....... ...	65.7	0.38	1.15	0.76	0.45	0.30	1.06	—0.68	0.53
July...........	69.2	2.13	2.35	1.55	1.45	0.96	2.51	—0.38	0.41
August........	66.2	3.06	1.95	1.29	1.60	1.06	2.35	0.71	0.42
September.....	56.2	2.40	0.88	0.58	1.09	0.72	1.30	1.10	0.54
October........	48.6	2.12	0.52	0.34	0.88	0.58	0.92	1.20	0.63
	43.4	22.40	8.49	5.61	10.17	6.73	12.34	10.06	11.36

(a) Precipitation from mean of thirteen U. S. Weather Bureau stations in and adjacent to the Wisconsin River water-shed above Merrill, Wis.

(b) Run-off by U. S. Geological Survey.

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TABLE 44.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area. Transpiration. From curve.	Loss from Land Area. Transpiration. Actual.	Loss from Land Area. Evaporation. From curve.	Loss from Land Area. Evaporation. Actual	Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
1910									
November	25.7	1.12	....	....	0.22	0.15	0.15	0.97	0.50
December	15.9	0.83	....	....	0.32	0.21	0.21	0.62	0.53
1911									
January	12.4	0.76	....	....	0.30	0.20	0.20	0.56	0.62
February	20.4	1.12	....	....	0.52	0.34	0.34	0.78	0.58
March	30.2	1.69	....	....	0.70	0.46	0.46	1.23	1.12
April	42.0	0.78	0.22	0.15	0.52	0.34	0.49	0.29	1.66
May	59.4	4.97	1.68	1.11	2.18	1.48	2.59	2.38	1.56
June	69.6	1.51	2.42	1.60	1.10	0.72	2.32	−0.81	0.81
July	67.8	7.05	2.30	1.52	3.30	2.18	3.70	3.35	0.64
August	63.8	4.72	1.80	1.19	2.11	1.39	2.58	2.14	1.00
September	56.6	4.17	1.20	0.79	1.68	1.11	1.90	2.27	1.29
October	43.4	6.52	....	....	1.70	1.12	1.12	5.40	3.79
	42.3	35.24	9.62	6.36	14.65	9.70	16.06	19.18	14.10
1911									
November	24.4	2.77	....	....	0.45	0.30	0.30	2.47	1.42
December	22.9	2.52	....	....	0.40	0.26	0.26	2.26	(1.80) a
1912									
January	−5.4	0.56	....	....	0.05	0.03	0.03	0.53	(1.20) a
February	11.4	0.27	....	....	0.28	0.18	0.18	0.09	(1.00) a
March	19.8	0.37	....	....	0.42	0.28	0.28	0.09	(1.00) a
April	42.1	3.06	0.22	0.15	1.15	0.76	0.91	2.15	3.05
May	55.2	4.89	1.40	0.92	2.00	1.32	2.24	2.65	2.78
June	61.6	1.78	1.88	1.24	1.08	0.71	1.95	−0.17	1.30
July	67.1	5.44	2.22	1.47	2.62	1.73	3.20	2.24	1.60
August	61.8	7.84	1.62	1.07	2.95	1.95	3.02	4.82	2.39
September	58.4	2.94	1.28	0.84	1.30	0.86	1.70	1.24	2.53
October	48.0	1.88	0.44	0.29	0.70	0.46	0.75	1.13	1.35
	38.9	31.32	9.06	5.98	13.40	8.64	14.82	19.50	21.42

(a) Estimated.

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TABLE 44.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.				Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
			Transpiration.		Evaporation.				
			From curve.	Actual.	From curve.	Actual.			
1912									
November......	33.1	0.62	....	....	0.18	0.12	0.12	0.50	0.98
December......	21.6	1.94	....	....	0.38	0.25	0.25	1.69	1.26
1913									
January........	14.0	0.59	....	....	0.35	0.23	0.23	0.36	0.94
February.......	8.4	1.02	....	....	0.18	0.12	0.12	0.90	0.84
March..........	21.6	2.61	....	....	0.68	0.45	0.45	2.16	1.18
April............	43.8	1.90	0.42	0.28	0.82	0.54	0.82	1.08	3.70
May............	52.0	4.06	1.08	0.71	1.70	1.12	1.83	2.23	2.03
June............	65.8	3.08	2.15	1.42	1.72	1.13	2.55	0.53	1.44
July............	65.6	6.75	2.15	1.42	3.02	1.99	3.41	3.34	2.00
August.........	66.4	2.48	1.97	1.30	1.38	0.91	2.21	0.27	1.15
September......	56.9	3.59	1.20	0.80	1.50	0.99	1.79	1.80	1.24
October.........	44.7	3.04	0.10	0.07	0.99	0.65	0.72	2.32	1.28
	41.0	31.68	9.07	6.00	12.94	8.50	14.50	17.18	18.04
1913									
November......	36.8	1.34	....	....	0.32	0.21	0.21	1.13	1.07
December......	28.3	0.14	....	....	0.21	0.14	0.14	0.00	0.88
1914									
January........	19.8	1.30	....	....	0.42	0.28	0.28	1.02	0.86
February.......	6.6	0.42	....	....	0.11	0.73	0.73	—0.31	0.73
March..........	24.5	1.65	....	....	0.62	0.41	0.41	1.24	0.75
April............	38.5	3.05	....	....	1.05	0.69	0.69	2.36	1.99
May............	56.5	2.19	1.48	0.97	1.15	0.76	1.73	0.46	1.89
June............	61.8	6.06	1.88	1.24	2.62	1.73	2.97	3.09	1.75
July............	71.1	4.66	2.50	1.65	2.50	1.65	3.30	1.36	1.57
August.........	64.6	5.94	1.89	1.25	2.60	1.72	2.97	2.97	1.44
September......	58.2	2.75	1.30	0.86	1.30	0.86	1.72	1.03	1.25
October.........	51.8	1.02	0.85	0.56	0.50	0.33	0.89	0.13	1.02
	43.2	30.52	9.90	6.53	13.40	9.51	16.04	14.48	15.20

Mr. Hoyt.

TABLE 45.—SUMMARY OF DATA AND COMPUTATIONS FOR WISCONSIN RIVER BASIN ABOVE MERRILL, WIS.

Drainage area, 2 630 sq. miles.

Year.	(a) Rainfall.	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	(b) Observed run-off.
1909........	25.89	7.62	5.93	13.55	12.34	14.90
1910........	22.40	6.73	5.61	12.34	10.06	9.01
1911........	35.24	9.70	6.36	16.06	19.18	......
1912........	34.32	8.84	5.98	14.82	19.50	19.52
1913........	31.68	8.50	6.00	14.50	17.18	17.55
1914........	30.52	9.51	6.53	16.04	14.48	15.22
Total.....	180.05	50.90	36.41	87.31	92.74	76.20
Mean......	30.00	8.48	6.07	14.55	15.45	15.24

(a) Precipitation from November 1st of preceding year to October 31st of given year.
(b) Run-off from March 1st of given year to February 28th or 29th of following year.

Mr. Hoyt.

TABLE 46.—Data and Computations for Wisconsin River Basin Between Merrill and Rhinelander, Wis.

Drainage area, 1 520 sq. miles.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	(a) Monthly precipitation, in inches.	Loss from Land Area. Transpiration. From curve.	Transpiration. Actual.	Evaporation. From curve.	Evaporation. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	(b) Observed run-off, in inches.
1908									
November......	32.6	1.47	....	....	0.25	0.15	0.15	1.32	0.67
December......	17.4	1.09	..	....	0.20	0.12	0.12	0.97	0.56
1909									
January........	15.4	0.53	....	....	0.42	0.24	0.24	0.29	0.47
February.......	17.6	1.26	....	....	0.50	0.29	0.29	0.97	0.43
March..........	25.0	1.29	....	....	0.62	0.36	0.36	0.93	0.50
April............	34.0	2.95	....	....	0.92	0.53	0.53	2.42	2.57
May.............	51.7	2.11	1.15	0.76	1.00	0.58	1.34	0.77	4.09
June............	65.2	3.36	2.10	1.39	1.90	1.10	2.49	0.87	1.66
July............	66.8	5.26	2.22	1.47	2.60	1.51	2.98	2.28	0.89
August.........	67.8	2.41	2.30	1.52	1.45	0.84	2.36	0.05	0.15
September......	57.4	2.45	1.20	0.79	0.15	0.67	1.46	0.99	0.42
October.........	43.1	1.71	....	....	0.52	0.30	0.30	1.41	0.46
	....	25.89	8.97	5.93	11.53	6.69	12.62	13.27	12.87
1909									
November......	37.0	4.48	....	....	1.05	0.61	0.61	3.87	2.00
December......	16.4	0.93	....	....	0.35	0.20	0.20	0.73	0.99
1910									
January........	13.8	0.76	....	....	0.38	0.22	0.22	0.54	0.71
February.......	11.4	0.97	....	....	0.25	0.14	0.14	0.83	0.52
March..........	40.0	0.28	....	....	0.42	0.24	0.24	0.04	1.37
April...........	46.8	2.46	0.72	0.48	1.10	0.64	1.12	1.34	2.16
May............	49.2	2.42	0.92	0.61	1.15	0.67	1.28	1.14	1.34
June............	65.7	0.38	1.15	0.76	0.45	0.26	1.02	—0.64	0.49
July............	69.2	2.13	2.35	1.55	1.45	0.84	2.39	—0.26	0.30
August.........	66.2	3.06	1.95	1.29	1.60	0.93	2.22	0.84	0.34
September......	56.2	2.40	0.88	0.58	1.09	0.63	1.21	1.19	0.50
October...... ..	48.6	2.12	0.52	0.34	0.88	0.51	0.85	1.27	0.64
	....	22.40	8.49	5.61	10.17	5.89	11.50	10.89	11.36

Mr. Hoyt.

TABLE 46.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area. Transpiration. From curve.	Transpiration. Actual.	Evaporation. From curve.	Evaporation. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
1910									
November......	25.7	1.12	....	....	0.22	0.19	0.19	0.93	0.44
December......	15.9	0.83	....	....	0.32	0.19	0.19	0.64	0.51
1911									
January........	12.4	0.76	....	....	0.30	0.17	0.17	0.59	0.46
February.......	20.4	1.12	....	....	0.52	0.30	0.30	0.82	0.47
March..........	30.2	1.69	....	....	0.70	0.41	0.41	1.28	1.23
April............	42.0	0.78	0.22	0.15	0.52	0.30	0.45	0.33	2.08
May.............	59.4	4.97	1.68	1.11	2.18	1.26	2.37	2.60	1.96
June............	69.6	1.51	2.42	1.60	1.10	0.64	2.24	− 0.73	0.88
July...........	67.8	7.05	2.30	1.52	3.30	1.91	3.43	3.62	0.25
August.........	63.8	4.72	1.80	1.19	2.11	1.22	2.41	2.31	0.39
September......	56.6	4.17	1.20	0.79	1.68	0.98	1.77	2.40	1.19
October........	43.4	6.52	....	....	1.70	0.99	0.99	5.53	4.51
	35.24	35.24	9.62	6.36	14.65	8.56	14.92	20.32	14.37
1911									
November......	24.4	2.77	....	....	0.45	0.26	0.26	2.51	1.70
December......	22.9	2.52	....	....	0.40	0.23	0.23	2.29	....
1912									
January........	—5.4	0.56	....	....	0.05	0.03	0.03	0.53	....
February.......	11.5	0.27	....	....	0.28	0.16	0.16	0.11	....
March..........	19.8	0.37	....	....	0.42	0.24	0.24	0.13	....
April............	42.1	3.06	0.22	0.15	1.15	0.67	0.82	2.24	3.67
May.............	55.2	4.89	1.40	0.92	2.00	1.16	2.08	2.81	3.56
June............	61.6	1.78	1.88	1.24	1.08	0.63	1.87	0.09	1.29
July............	67.1	5.44	2.22	1.47	2.62	1.52	2.99	2.45	1.76
August.........	61.8	7.84	1.62	1.07	2.95	1.71	2.78	5.06	1.90
September......	58.4	2.94	1.28	0.84	1.30	0.75	1.59	1.35	2.61
October........	48.0	1.88	0.44	0.29	0.70	0.41	0.70	1.18	1.07
	......	34.32	9.06	5.98	13.40	7.77	13.75	20.57	....

Mr.
Hoyt.

TABLE 46.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	LOSS FROM LAND AREA. Transpiration. From curve.	Transpiration. Actual.	Evaporation. From curve.	Evaporation. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
1912									
November	33.1	0.62	....	....	0.18	0.10	0.10	0.52	0.42
December......	21.6	1.94	....	....	0.38	0.22	0.22	1.72	1.06
1913									
January........	14.0	0.59	....	....	0.35	0.20	0.20	0.39	1.06
February.......	8.4	1.02	....	....	0.18	0.10	0.10	0.92	0.80
March...........	21.6	2.61	....	....	0.68	0.39	0.39	2.22	1.16
April	43.8	1.90	0.42	0.28	0.82	0.48	0.76	1.14	4.90
May	52.0	4.06	1.08	0.71	1.70	0.99	1.70	2.36	2.31
June............	65.8	3.08	2.15	1.42	1.72	1.00	2.42	0.66	1.56
July.............	65.6	6.75	2.15	1.42	3.02	1.75	3.17	3.58	2.47
August	66.4	2.48	1.97	1.30	1.38	0.80	2.10	0.38	1.23
September	56.9	3.59	1.20	0.80	1.50	0.87	1.67	1.92	1.28
October.........	44.7	3.04	0.10	0.07	0.99	0.57	0.64	2.40	0.93
	......	31.68	9.07	6.00	12.94	7.47	13.47	18.21	18.98
1913									
November......	36.8	1.34	....	....	0.32	0.19	0.19	1.15	0.70
December	28.3	0.14	....	....	0.21	0.12	0.12	0.02	0.59
1914									
January........	19.8	1.30	....	....	0.42	0.24	0.24	1.06	0.65
February.......	6.6	0.42	....	....	0.11	0.06	0.06	0.36	0.51
March..........	24.5	1.65	....	....	0.62	0.36	0.36	1.29	0.51
April............	38.5	3.05	....	....	1.05	0.61	0.61	2.44	2.52
May	56.5	2.19	1.48	0.97	1.15	0.67	1.64	0.55	2.35
June............	61.8	6.06	1.88	1.24	2.62	1.52	2.76	3.30	2.20
July............	71.1	4.66	2.50	1.65	2.50	1.45	3.10	1.56	1.28
August	64.6	5.94	1.89	1.25	2.60	1.51	2.76	3.18	1.12
September......	58.2	2.75	1.30	0.86	1.30	0.75	1.61	1.14	1.09
October.........	51.8	1.02	0.85	0.56	0.50	0.46	1 02	0.00	1.19
	......	30.52	9.90	6.53	13.70	7.94	14.47	16.05	14.71

Mr. Hoyt.

TABLE 47.—SUMMARY OF DATA AND COMPUTATIONS FOR WISCONSIN RIVER BASIN BETWEEN RHINELANDER AND MERRILL, WIS.

Drainage area, 1 520 sq. miles.

Year.	(a) Rainfall.	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	(b) Observed run-off.
1909.......	25.89	6.69	5.93	12.62	13.27	14.96
1910.......	22.40	5.89	5.61	11.50	10.90	9.02
1911.......	35.24	8.56	6.36	14.92	20.32	....
1912.......	34.32	7.77	5.98	13.75	20.57	19.70
1913.......	31.68	7.47	6.00	13.47	18.21	18.29
1914.......	30.50	7.94	6.53	14.47	16.05	14.33
Total.....	......	......	......	......	79.40	76.30
Mean......	......	......	......	......	(c) 15.88	15.26

(a) Precipitation from November 1st of preceding year to October 31st of given year.
(b) Run-off from March 1st of given year to February 28th or 29th of following year.
(c) Not including 1911, for which there were no corresponding run-off data.

Mr. Hoyt.

TABLE 48.—Run-Off, in Second-Feet per Square Mile, for a Number of Wisconsin Rivers.

November 20th, 1914, to March 10th, 1915.

River.	Station.	Nov., 1914.	Dec., 1914.			Jan., 1915.			Feb., 1915.			Mar., 1915.
		20-30	1-10	11-20	21-31	1-10	11-20	21-31	1-10	11-20	21-28	1-10
Lake Superior Basin.												
Aminicon........	Aminicon Falls.	0.25	0.20	0.18	0.16	0.14	0.12	0.06	0.06	0.07	0.09	0.24
Bad..............	Odanah..........	0.45	0.48	0.26	0.18	0.18	0.21	0.16	0.16	0.23	0.28	0.29
Average, Lake Superior Basin....		0.35	0.34	0.22	0.17	0.16	0.16	0.11	0.11	0.15	0.18	0.26
Mississippi River Basin.												
Namakagon.....	Trego............	0.95	0.88	0.82	0.61	0.61	0.69	0.60	0.62	0.67	0.66	0.95
St. Croix.........	Swiss............	0.97	0.97	0.75	0.60	0.56	0.55	0.52	0.55	0.56	0.66	0.60
St. Croix........	St. Croix Falls..	0.34	0.44	0.25	0.26	0.32	0.30	0.27	0.26	0.32	0.32	0.32
Chippewa........	Lessard's.........	0.50	0.51	0.46	0.44	0.50	0.48	0.41	0.40	0.48	0.56	0.58
Chippewa........	Bishop's Bridge.	0.57	0.56	0.58	0.44	0.48	0.43	0.41	0.41	0.44	0.48	0.54
Chippewa........	Bruce............	0.58	0.64	0.58	0.44	0.42	0.46	0.40	0.37	0.47	0.49	0.32
Chippewa........	Chippewa Falls.	0.38	0.48	0.40	0.30	0.26	0.23	0.24	0.26	0.27	0.32	0.32
Flambeau.......	Butternut........	0.70	0.75	0.68	0.60	0.65	0.64	0.63	0.58	0.58	0.59	0.59
Flambeau.......	Ladysmith	0.54	0.45	0.34	0.35	0.25	0.25	0.32	0.36	0.53	0.44	0.44
Eau Claire......	Augusta..........	0.40	0.38	0.28	0.16	0.12	0.11	0.11	0.16	0.36	0.84	0.32
Trempealeau....	Dodge............	0.38	0.42	0.37	0.36	0.37	0.38	0.22	0.38	0.78	1.70	0.66
Black............	Neillsville........	0.14	0.25	0.03	0.05	0.08	0.09	0.05	0.04	0.12	0.49	0.18
Average, Mississippi River Basin..		0.54	0.53	0.45	0.38	0.38	0.37	0.35	0.37	0.47	0.62	0.48
Wisconsin River Basin.												
Wisconsin.......	Rhinelander.....	0.95	0.92	0.62	0.70	0.72	0.73	0.76	0.76	0.77	0.84	1.04
Wisconsin.......	Muscoda.........	0.49	0.50	0.45	0.40	0.38	0.40	0.38	0.41	0.72	1.15	0.84
Prairie...........	Merrill...........	0.86	0.76	0.52	0.46	0.43	0.46	0.48	0.52	0.59	0.62	......
Little Rib.......	Wausau..........	0.12	0.18	0.10	0.08	0.06	0.09	0.10	0.12	0.45	0.52	0.10
Eau Claire......	Kelly.............	0.30	0.42	0.22	0.20	0.20	0.19	0.19	0.22	0.25	0.34	0.29
Big Eau Pleine..	Stratford........	0.12	0.10	0.04	0.02	0.01	0.01	0.01	0.02	0.04	0.23	0.14
Plover...........	Stevens Point...	1.00	1.10	0.85	0.72	0.58	0.52	0.49	0.58	0.73	0.70	0.85
Baraboo.........	Baraboo.........	0.26	0.25	0.26	0.18	0.20	0.44	0.33	0.38	0.99	2.30	0.63
Average, Wisconsin River Basin...		0.51	0.54	0.39	0.35	0.31	0.35	0.33	0.38	0.57	0.84	0.54
Rock River Basin.												
Rock.............	Afton............	0.30	0.34	0.28	0.24	0.22	0.25	0.27	0.34	.....	.....	......
Sugar............	Brodhead........	0.35	0.48	0.42	0.29	0.27	0.38	0.44	0.43	1.46	.....	......
Pecatonica	Dill...............	0.35	0.34	0.33	0.32	0.30	0.28	0.26	0.50	0.95	.....	......
Average, Rock River Basin.......		0.33	0.39	0.34	0.28	0.26	0.26	0.32	0.42	1.20	.....	......
Lake Michigan Basin.												
Menominee......	Koss.............	0.66	0.78	0.42	0.38	0.40	0.43	0.42	0.40	0.46	0.48	0.46
Brule............	Florence.........	0.92	0.88	0.78	0.74	0.70	0.62	0.52	0.55	0.66	0.82	0.78
Pine.............	Florence.........	0.76	0.69	0.52	0.44	0.42	0.39	0.32	0.32	0.38	0.45	0.42
Pike.............	Amberg..........	0.84	0.72	0.55	0.54	0.56	0.57	0.56	0.58	0.66	0.62	0.60
Oconto..........	Gillett...........	0.82	0.89	0.75	0.55	0.52	0.50	0.55	0.56	0.60	0.62	0.64
Wolf.............	New London....	0.62	0.66	0.48	0.38	0.34	0.35	0.38	0.36	0.54	0.83	0.72
Average, Lake Michigan Basin....		0.77	0.77	0.58	0.51	0.49	0.48	0.46	0.46	9.55	0.64	0.59

Adolph F. Meyer,* M. Am. Soc. C. E. (by letter).—The writer has read with considerable interest the various discussions of his paper. It was a source of some gratification to find general agreement with practically all the important principles underlying the writer's method. The only one who appears to differ in any material respect is Mr. Justin. Although practically all his contentions are either directly or incidentally met, and his criticisms answered, by Messrs. Chandler, Horton, and Hoyt, a few additional statements in reply may not be amiss. In the first place, Mr. Justin contends that the writer has not given proper recognition to the effect of slope on run-off. As pointed out by Mr. Horton, however, slope, as a factor in determining run-off, is of less importance than might at first appear. Perhaps even Mr. Justin, in discussing the paper, has over-estimated the weight given to the slope factor in his (Mr. Justin's) formula for run-off, given on page 1275.† In that formula, the factor, S, representing the slope of the water-shed (derived by the illogical procedure of dividing the difference between the highest and the lowest points on the water-shed by the square root of the area, without reference to the ruggedness of the country), appears with an exponent of 0.155, which is less than the one-sixth power. On the other hand, R, representing the annual rainfall, is raised to the second power, and T, representing the mean annual temperature, appears in the denominator with the first power. In other words, Mr. Justin considers rainfall about thirteen times as important a factor as slope. It is very evident, then, that even Mr. Justin does not, in his formula, give to the slope factor the importance which one would infer is attached to it from his criticism of the paper.

Mr. Meyer.

The writer takes exception to Mr. Justin's interpretation of his comments relating to the necessity for accurate estimates of the monthly distribution of run-off. Accuracy, in this connection, as in most others, is a relative matter. The writer does not subscribe to Mr. Justin's interpretation of desired accuracy in monthly distribution of run-off, merely because even a rather approximate monthly distribution does not lead to gross errors in determining storage from a mass-curve constructed on such a small scale as to preclude the possibility of accurate reading. Moreover, he does not agree that Mr. Justin has shown, by his use of the mass-curve, that monthly temperatures need not be taken into account in estimating monthly run-off, in fact, he desires to reiterate most emphatically the opposite view.

Mr. Justin next credits the writer with over-emphasizing the importance of the long-term mean as against the long-term record, on pages 631 and 632.‡ Referring to these pages one reads:

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* Minneapolis, Minn.

† *Proceedings*, Am. Soc. C. E., for May, 1915.

‡ *Proceedings*, Am. Soc. C. E., for March, 1915.

Mr. Meyer. "On comparatively few streams of the country do the records of discharge extend over a long term of years. Short-term records do not give the extremes of high and low flow unless by sheer accident such years have been included in the term over which observations extend. Short-term records, moreover, do not give a satisfactory value for mean utilizable flow. In the last analysis, it is usually necessary to supplement the observed stream-flow data with computed values based on rainfall and other physical data, in order to arrive at a probable maximum, minimum, and mean utilizable flow for any given stream.

"In order to show the annual and periodic variations in rainfall and run-off on the two streams, considered in this paper, for which relatively long-term records are available, the curves in Figs. 34 to 36 have been prepared."

This quotation by itself appears to reply to Mr. Justin's criticism.

Mr. Justin next refers to the writer's subdivision of losses out of rainfall into evaporation losses from land and water surfaces and transpiration losses, and comments on this subdivision as being "both an unfortunate and unnecessary complication of the subject". Here, again, the writer's position is fully approved by the discussions of Messrs. Chandler, Horton, and Hoyt. Mr. Justin has either failed to read or has misconstrued, the writer's discussion of the subject of transpiration, particularly in that he concludes that all the experiments of the United States Department of Agriculture fail to differentiate between transpiration and evaporation. In reference to this matter, Mr. Justin is respectfully referred, in particular, to *Bulletin 285*, Bureau of Plant Industry, on "The Water Requirements of Plants", which gives a full review of the literature on the subject.

Mr. Justin next refers to the possibility of deriving a formula for the transpiration of any given water-shed by representing the following factors mathematically,

"Thriftiness of inhabitants (as a coefficient to be determined by judgment); tons of hay, bushels of corn, oats, rye, wheat, etc., density of population per square mile, percentage of farmers who are graduates of agricultural colleges, tons of fertilizer per acre, gallons of alcohol consumed per capita per year, etc., etc."

Such views require no comment.

Mr. Justin then refers to the transpiration curve, commenting,

"Although the author does not particularly emphasize the fact, it is clear that this curve is not based on observed data, but on the assumption, that 'the law first stated by Van't Hoff and Arrhenius, that most chemical reactions and physiological processes double in activity for every rise in temperature of 10° cent.' is also applicable to transpiration."

If Mr. Justin would take time to investigate, he could readily determine that the law of Van't Hoff and Arrhenius is not a mere

assumption. If he will even refer to the top of page 583* of the paper, he will find at least one reference to the verification of this law in connection with transpiration phenomena. Mr. Meyer.

Those who have given the paper an unbiased reading, or who have had some previous knowledge of the subject of transpiration, will hardly agree with Mr. Justin's sweeping conclusion, stated in these terms:

"Thus, it is demonstrated that the differentiation between evaporation from ground surfaces and transpiration is a matter which is still in the realm of surmise."

Perhaps the best measure of the manner in which Mr. Justin has apparently perused the author's paper is afforded by the following quotation:

"When the author compiled Table 5, he had before him the recorded rainfall and run-off for the various water-sheds considered. He then manipulated his curves, his coefficient, and his judgment to derive the three quantities, evaporation from water surfaces, evaporation from ground surfaces, and transpiration, always bearing in mind the fact, subconsciously or otherwise, that the sum of the three must approximately equal the difference between run-off and rainfall. Nothing could be more simple."

Such insinuations of the juggling of figures, as those made by Mr. Justin in this paragraph, are no more worthy of comment than his remarks respecting the consideration of such matters as educational training and alcohol consumption in connection with the determination of transpiration from a given water-shed, for the purposes of estimating stream flow.

If Mr. Justin still believes that the intricate relationship between such natural phenomena as rainfall and run-off can be expressed by a simple mathematical formula, he is welcome to that belief. In his paper on "Derivation of Run-Off from Rainfall Data",† however, he did not prove that such a simple relationship existed, even though his formula gave reasonably accurate results on the limited number of water-sheds, having a relatively constant annual rainfall of from 35 to 50 in., to which it was applied. The writer has applied his method, with good results, over a considerably greater range of natural phenomena. A formula of the kind used by Mr. Justin can never give more than approximate results, and then only on water-sheds where the rainfall is considerably more than sufficient to supply all ordinary evaporation and transpiration losses. The writer is supported in this view by several of the other engineers who have discussed the paper, and by the opinions of hydraulic engineers as recently expressed in other technical literature.

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* *Proceedings*, Am. Soc. C. E., for March, 1915.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 346.

Mr. Meyer. Mr. Justin will find, on page 611,* at the foot of Table 7, in which the column heading "Actual Evaporation" is first used, a note stating exactly what this column headed "Actual Evaporation" refers to, without finding it necessary to warn those who actually have read, or will read, the paper that this column represents "merely the author's estimate", and not actual measured evaporation.

Mr. Strong's discussion is interesting and to the point, and requires no special comment. The emphasis laid on judgment, experience, and a thorough knowledge of the laws governing evaporation, transpiration, and underground flow, combined with a knowledge of the physical data for the drainage area under consideration, in order to be able to make rational estimates of run-off, is timely, and entirely in accord with the writer's view.

Mr. Chandler evidently agrees with the writer's chain of reasoning, and with the fundamental principles involved in his method of computing run-off. It is to be regretted that Mr. Chandler did not have time to complete the tabulations involved in an application which he appears to have made, of the writer's method, to several Dakota streams.

The writer is entirely in accord with the second point to which Mr. Chandler calls attention. Better results will be secured in every case, particularly on those water-sheds where the rainfall is less than 25 in., on southern water-sheds where the losses are high, and on all water-sheds having considerable portions which differ widely in physical characteristics, by subdividing the drainage basin above the point at which estimates of run-off are desired. If rainfall and run-off bore a direct relation to each other, there would be no harm in averaging the monthly rainfall records for a large water-shed. It is contrary to the fundamental principles underlying the writer's method, however, to average greatly varying quantities of precipitation for the purpose of deducing an average for a large water-shed. In fact, he desires to confess that he made the rough application of his method to the large water-shed of the Colorado River at Austin, Tex., more from curiosity than with the expectation of deriving rational results. While making the computations for this water-shed, and noting where the computed precipitation minus losses were negative and where, nevertheless, considerable surface run-off appeared in the stream, he frequently had occasion to refer to the precipitation records and to note that in such instances there invariably was excessive precipitation on one or more minor tributaries of the stream, and little or no precipitation over the drainage basins of other tributaries.

The writer also agrees with the third point made by Mr. Chandler, on page 1278.† He desires to add, however, that even the discharge

* *Proceedings*, Am. Soc. C. E., for March, 1915.

† *Proceedings*, Am. Soc. C. E., for May, 1915.

records of the United States Geological Survey (at least for Minnesota streams) do not take into consideration lake storage, but give merely the run-off from the water-shed above the station, even though such run-off represents little else than outflow from a lake. Failure to take lake storage into consideration has led to grossly inaccurate conclusions, with respect to the possibility of reservoir control of stream flow, in several mass-curve studies with which the writer is familiar. Mr. Meyer.

The only important point of criticism made by Mr. Chandler with which the writer does not entirely agree, is the statement on page 1280* to the effect that as small an error as 5% in the assumed evaporation coefficient might double or treble the computed run-off, or might wipe it out entirely, in such regions as the Northwestern Central States. Such a result as that feared by Mr. Chandler would be secured only in case the transpiration losses were not properly reduced during months of deficient rainfall and deficient storage of water in the upper few feet of soil, or on those water-sheds from which the normal run-off is only about 1 in. or less. The writer has frequently expressed his firm belief in the necessity for meter measurements of the discharge of streams, particularly in the case of streams in which the annual discharge, measured in inches of depth on the tributary water-shed, is exceedingly small.

Mr. Grunsky points out the surprising uniformity in the quantity of run-off from a water-shed on which the rainfall approximates or exceeds 40 in. per annum. He presents a curve (Fig. 37) based on a relatively simple and direct relationship between rainfall and run-off. Although this curve may give satisfactory results for the California conditions under which it was derived, its application must of necessity be limited, because it fails to recognize, in an adequate manner, the various factors which determine the run-off from any given water-shed.

Mr. Horton has given an interesting and instructive summary of factors modifying stream flow, and of different methods at present in use for computing run-off from a given drainage basin.

It is gratifying to find an engineer of Mr. Horton's standing coming forward with a clear-cut statement in favor of a method of estimating run-off which takes into consideration, not only rainfall, but other physical data pertaining to the drainage basin, such as temperature and the physical characteristics and cultural conditions of the water-shed.

Perhaps the writer failed to state with sufficient clearness, or perhaps Mr. Horton misinterpreted the reference to the results of the experiments of Transeau, given on page 578.† Mr. Horton points out that these experiments appear to indicate greater evaporation from

* *Proceedings*, Am. Soc. C. E., for May, 1915.

† *Proceedings*, Am. Soc. C. E., for March, 1915.

Mr. Meyer. a bare gravel slide than from any other condition of the soil. The writer's statement, introducing these experimental data, is as follows:

"It is a well-known fact that all forms of vegetation, particularly forests, shade the ground to a certain extent, and consequently reduce the rate of evaporation of free moisture. Whether or not they reduce the total quantity evaporated per month, or per year, depends also on the relative rates at which the rainfall can percolate into the forest floor, the cultivated field, and the bare ground, or run off into the streams."

It appears to the writer that this statement anticipates Mr. Horton's remark to the effect that:

"Actually, the evaporation, particularly for the gravel slide, is of short duration, and in some cases is continuous for the other soil conditions cited. The rate may be greater, but the total evaporation is less for the gravel slide than for the other conditions."

This statement is entirely in accord with the writer's views just quoted, and repeated in other portions of the paper, as for example, on pages 574 and 579.*

It is to be regretted that Mr. Horton did not present more of the details of his method of estimating run-off. It is noted that he further subdivides rainfall losses by using the factor "interception" loss. In the writer's method, interception is taken account of in the evaporation coefficient, as indicated on pages 578 and 579.* In the case of water-sheds covered with a dense growth of coniferous trees, the writer also uses a larger evaporation coefficient during the winter, when the precipitation occurs as snow, than during the summer, and larger, also, than would be used in case the water-shed were covered with deciduous trees.

Mr. Horton's comments as to the lack of effect of slope on the interception loss is in accord with the writer's views. "Evaporation opportunity" appeals to the writer as an apt expression.

Mr. Horton expresses the opinion that the writer's method does not adequately cover or account for transpiration losses. This may be entirely true, yet the basis for improvement does not appear to be afforded by Mr. Horton's discussion. "Crop yield" is referred to as the best and simplest measure of transpiration loss generally available. This may be substantially true, when considering the average transpiration over a period of years from cultivated water-sheds, and to this extent is utilized by the writer in determining the normal seasonal transpiration (pages 586 and 595*). Crop yields, of course, are of little assistance in estimating transpiration from timbered or brush-covered water-sheds. Crop yields, in the case of grain, also fail to take into account differences in the yield of straw, which may result in

* *Proceedings,* Am. Soc. C. E., for March, 1915.

very substantial differences in the transpiration loss. The relation between transpiration loss and vegetable matter produced is frequently referred to in the paper; nevertheless, monthly rainfall and temperature, and quantity of moisture stored in the soil, are believed to afford a better practical index to monthly transpiration losses than crop yields, on most water-sheds in the United States. Mr. Meyer.

The writer was very much pleased to find that at least one engineer felt that it was necessary to make an actual application of the writer's method of computing run-off in order to be able to pass fair judgment, and has presented the detailed computations involving this application as a portion of his discussion. Although the results of Mr. Hoyt's computations bear minor modification, they nevertheless indicate that reasonably satisfactory estimates of run-off may be made by this method, by any engineer who takes the necessary time to familiarize himself with it even as briefly presented in the paper.

Mr. Hoyt states that, in the introduction to recent papers on the surface water supply of the United States Geological Survey, a note has been added, calling attention to the fact that the figures showing depth of run-off, in inches on the water-shed, may be subject to gross errors resulting from the inclusion of large non-contributing districts, or from the omission of estimates of water diverted for irrigation, or other purposes, from a portion of the water-shed, and that consequently, run-off values have not been stated in inches of depth on the drainage area for all streams having water-sheds on which the annual rainfall is less than 20 in., nor on drainage areas for which such figures of run-off might be misleading on account of stream flow diversion, etc. The writer agrees entirely with the Geological Survey in the desirability of adding this warning. It is well to have in mind, however, that the daily discharge records, as published by the Geological Survery, of the streams to which this warning applies, merely constitute a report of what has happened in the past, and offer no adequate indication of what may occur in the future at the given point on the stream, or on its tributaries. If the work of stream gauging is to be confined to an historical study, then such published data of stream discharge are valuable as history, but if the results of stream gauging are to be used as the basis for the prediction of the quantities of water which will probably be available from month to month during succeeding years, at different points on the basin of which the run-off was gauged, then some basis for making extensions and analyses of records must be found. For example, the basic principle underlying the control which is being exercised over the flow of the stream, by the given reservoirs, must be determined. The extent and location of present diversions from the water-shed must be known with at least reasonable accuracy, and estimates of future diversions must be made. The uses to which the diverted water is

Mr. Meyer. put must be known, so that estimates may be made of the probable future diversion from month to month under varying quantities of monthly precipitation. If large portions of a drainage area are non-contributing under certain meteorological conditions, this fact must be determined, and the reason for changes in the quantity of water contributed by certain portions of the water-shed must be at least reasonably well determined. The writer contends that, even on such water-sheds as those referred to in the note quoted by Mr. Hoyt from the recent Water Supply Papers, estimates must be made of probable future yields of water. Such estimates can only be made on the basis of past records of stream flow, properly analyzed in connection with all the available physical data for the given water-shed.

In the writer's estimation, the records of daily discharge published by the Geological Survey, for such streams as those just referred to and which, for the reasons given, are not reduced to inches of depth on the water-shed, are of little value as the basis for expenditures for works of improvement which are in any way dependent on stream flow, until these records of discharge have been given detailed study and analysis in connection with physical data for the given water-shed. Certainly, past records of stream flow are of no help in determining the probable future yield of water from the given drainage basin until the hydrological phenomenon and physical conditions which have resulted in the given observed yields, have become known.

The very essence of the writer's method of computing run-off from rainfall and other physical data is a careful study of the physical characteristics of the given drainage basin. Where the portion of the drainage basin of any given stream, which does not ordinarily contribute to the run-off, forms a large percentage of the entire drainage basin above the point at which estimates of run-off are desired, such a drainage basin should be subdivided, and estimates should be made of the probable run-off from the several "homogeneous" portions of the basin, by a detailed study of each, instead of by averaging precipitation, temperature, cultural, and other physical conditions over the entire water-shed, and applying a single coefficient. In reference to this, however, it should be noted that in the paper a study was made of three California streams having drainage basins which differed widely. The method was applied to two tributaries, and then to the parent stream, which included the tributaries for which run-off was separately estimated, and though the coefficients of evaporation for the two tributaries were 0.60 and 1.10, respectively, and the coefficient used for the parent stream was 0.85, the computed annual precipitation minus loss for both the tributaries and the parent stream compares surprisingly well with the observed run-off, indicating, though it does not prove, that even where it is necessary to average widely different physical characteristics on a given water-shed, and

Mr. Meyer.

to use a single coefficient for a relatively large water-shed, it may, nevertheless, be possible to secure surprisingly close estimates of run-off, from rainfall and other physical data, by the writer's method.

The writer desires to emphasize the point made by Mr. Hoyt, that though a run-off year beginning on March 1st corresponds very well with a rainfall year beginning on November 1st, on a number of the streams of Minnesota studied by the writer, these water years are not applicable to streams on which any substantial quantity of surface run-off occurs during the winter. On such streams, a water year beginning about September 1st is more satisfactory.

Mr. Hoyt desires to know why the writer did not use the evaporation observations made at Madison and Menasha, Wis., and at Iowa City, Iowa. The only indication of the reason for the omission of the records to which Mr. Hoyt refers is given on page 570* in the following words:

"Many other records of evaporation are available, but are omitted here because they do not give the evaporation from shallow water under conditions of wind and humidity prevailing throughout the Northwest."

The observations of evaporation made at Menasha, Madison, and Iowa City give the evaporation from relatively large, deep bodies of water. If Mr. Hoyt will plat these records on the curve of evaporation from water, snow, and ice, Fig. 12, which gives the evaporation from open bodies of water of medium size and depth, he will find that the records at the stations referred to agree more nearly with this curve than with the curve of evaporation from shallow water. This curve of evaporation from shallow water, Fig. 8, was used in connection with the construction of the curves of evaporation from land areas. In view of the fact that the temperature of all relatively large or deep bodies of water is considerably lower than that of the air, as observed by the Weather Bureau, during the spring, and higher during the fall, it would clearly have been wrong to have used any records of evaporation from other than shallow bodies of water, in connection with the determination of the evaporation of water from land areas, the temperature of which more nearly approximates the air temperature as observed by the Weather Bureau.

Mr. Hoyt requests more detailed information in regard to the development of the curve of evaporation from land areas. Most of the basic facts and considerations are presented in the paper, and there is little more to be said, except that the curve represents the result of a number of revisions following trial applications of the method to typical water-sheds. The curves have gone through four or five revisions, and those presented were adopted because they gave the best results when actually applied in estimating stream flow.

It is very true, as Mr. Hoyt states, that the run-off resulting from a given quantity of rain falling in short intervals, differs radi-

* *Proceedings*, Am. Soc. C. E., for March, 1915.

Mr. Meyer. cally from that resulting from well-distributed rains, yet it is also true that the normal distribution of rainfall, when the month is used as a basis, is reasonably similar, year after year, in any given locality and for the same month of the year. These ordinary conditions are taken account of in arriving at the proper evaporation coefficient. Extraordinary or excessive rainfalls must be given separate consideration. The writer's method, as presented, furnishes, primarily, a skeleton of basic principles and steps in the computation of run-off. There is practically no limit to the degree of refinement to which the computations may be carried by taking into account, in the computation of losses for each month, variations from the normal meteorological phenomena for the given water-shed. Whenever warranted by the importance of the results to be secured, daily records of temperature and precipitation should be used to assist in computing monthly evaporation and transpiration losses.

Mr. Hoyt regrets that the writer has not given the transpiration and evaporation coefficients used for the several drainage basins to which the method of computing run-off has been applied. The "evaporation coefficients" are given in Table 5. No transpiration coefficients are given because the transpiration, as taken from the transpiration curve, must be further corrected, as indicated in the outline of the method on pages 594 to 596,* under "II-C-3", for deficient moisture supply.

In Mr. Hoyt's application of the writer's method, no corrections have apparently been made for deficient rainfall and soil storage in arriving at the monthly transpiration. For example, on the Rock River water-shed, in September, 1908, Mr. Hoyt used the full transpiration loss of 1.59 in., which is almost exactly equal to the precipitation for that month, and which resulted in a negative "precipitation minus loss" for that month of more than 1 in. September followed 3 months of deficient precipitation, the losses for July being credited with more than 1 in. of moisture derived from soil storage. Now, it stands to reason that if the rainfall for every month, since May, has been insufficient to supply the normal requirements of evaporation and transpiration, there will not be sufficient moisture available in the soil to permit full normal transpiration during September, even if plant growth had not been stunted as the result of insufficient rainfall during the entire season. The transpiration for that season should have been reduced at least from 1 to 1.25 in. The computed annual precipitation minus losses for this water-shed during 1907 should also have been reduced by about 1 to 1.25 in., and that for 1908 should have been increased by a similar quantity on account of evident changes in ground storage.

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* *Proceedings*, Am. Soc. C. E., for March, 1915.

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The writer is unable to furnish any further information with respect to the coefficients used on the several water-sheds treated in the paper. The coefficient adopted for each water-shed was used throughout the entire period of years over which run-off was computed, consequently there is no such variation from year to year as one would infer from Mr. Hoyt's comments. If the coefficient had been varied from year to year, it would, of course, have been possible to have made the computed run-off agree exactly with the observed. All inaccuracies in the method are thrown into the final result, which is the computed annual run-off, and has been placed beside the observed annual run-off, so that the equivalent of the information which Mr. Hoyt requests is already given in the paper.

Mr. Hoyt gives a table of coefficients (Table 36), which he selected because they gave the most consistent results. The writer has never attempted to go to the refinement of expressing coefficients to single hundredths, such as 0.59, used by Mr. Hoyt. In this case the writer would use 0.60, and instead of 0.66 he would use 0.65, because the use of coefficients stated to single hundredths is not warranted even by the run-off data from which Mr. Hoyt derived his values. As a general criticism, the writer would state that Mr. Hoyt used slightly larger transpiration losses than would be indicated by the values given in the paper for approximately similar cultural and light conditions.

The writer believes that Mr. Hoyt's Table 37, indicating the coefficient which it would have been necessary to apply to the combined values as taken from the evaporation and transpiration curves in order to secure the necessary loss so as to leave a precipitation minus loss equal to the observed run-off, offers no information which affords any criterion by which to judge the accuracy of the writer's method. The variation in coefficients indicated is particularly misleading, as all changes due to soil and subsoil storage are thrown into the coefficient. The proper basis for comparison is computed with observed run-off, using the same coefficient throughout the entire period of years, unless radical changes in cultural conditions occurred on the water-shed.

Mr. Hoyt apparently has misinterpreted, or perhaps the writer has failed to state with sufficient clearness, the ordinary range of evaporation coefficients for the Northwest. In reference to the evaporation coefficient, it is stated (page 593*), "This coefficient ranges from about 0.95 to 1.25 for most water-sheds of the Northwest, and for similar ones elsewhere". Then the factors on which the coefficient depends are summarized, and then there is the statement: "Between these extremes the usual working values will be found". Evidently 0.95 to 1.25 represent usual working values for this coefficient "for most water-sheds of the Northwest, and for similar ones elsewhere".

* *Proceedings*, Am. Soc. C. E., for March, 1915.

Mr. Meyer. The three Wisconsin River water-sheds considered by Mr. Hoyt are so sandy, according to the descriptions quoted from Mr. Stewart, as to permit no surface run-off during the open season; hence they are clearly of an exceptional character. The values, 0.95 to 1.25, as stated before, represent ordinary working values for most water-sheds of the Northwest. They do not represent the coefficients which would be used for either a sand pit or a city pavement. It is entirely unnecessary for the writer to find "causative errors" in Mr. Hoyt's computations, neither is it "unfortunate" that the sandy water-sheds of the northern Wisconsin River Basin require the use of a small evaporation coefficient.

If the writer gave the impression, as one might gather from Mr. Hoyt's statements on page 1963, that his method of computing monthly run-off was "largely a matter of judgment in adding to and subtracting from the 'precipitation minus losses', as determined from the various curves", it was unfortunate, because quite contrary to the facts.

The paper presents three curves which form the basis for estimating the monthly distribution of run-off for the Root River watershed, and gives the detailed computations for this water-shed so that the method may be clear. It would appear, from the computations presented for this water-shed in Table 35, that, for a considerable portion of the year, the stream flow consists entirely of seepage. The determination of seepage flow, however, involves no estimate whatever, the values being taken directly from the curve of Fig. 32. Even if these three sets of curves were rigidly applied, the result would still be a rational monthly distribution of run-off, although the quantities secured in that way would not agree as well with the observed run-off as the writer's computations, which take daily temperature, precipitation, etc., into consideration, as indicated by the column, in Table 35, headed "Notes".

It is impossible to give detailed information regarding the development of these curves of Figs. 30, 31, and 32, because they are not based on any definite group of data, but on a process of logical reasoning, experience, observation, and all the facts bearing on the subject which the writer could command. These curves would necessarily have to be changed before being applicable to different streams, although the change would be small for some water-sheds. On a steeper, more impervious water-shed, for example, the percolation curve in Fig. 30, which now has a value of 2 in., might be given a value of 0, and the curves having values of 3 and 4 in., respectively, might receive values of 1 and 2 in., although these curves would be more closely spaced. In a similar manner, the curves of Fig. 31 would receive different values. For a perfectly impervious, sloping watershed, the curve of zero soil storage would be approximately in the position of the curve of 4 in. soil storage in this figure.

Mr. Meyer.

The frequency curves presented by Mr. Hoyt are not frequency curves of computed monthly run-off, as one would infer from his statements. They merely represent monthly precipitation minus losses, and, as such, are interesting in that they show the equalizing effect of soil and surface storage on stream flow. Even though not intended for this purpose, these curves are perhaps the best graphical proof of the lack of a simple, direct percentage relationship between precipitation and run-off that could be presented, and all believers in exponential formulas for determining run-off from rainfall are respectfully referred thereto.

Frequency curves of computed monthly run-off and observed monthly run-off, for the Root River, for example, are almost identical when covering the same period of time, although the frequency curve of computed monthly run-off, based on 20 years of rainfall and other data, is radically different from that based on the short-term observed run-off records. This, again, emphasizes the need for supplementing and extending short-term observed stream flow records.

The writer does not hold that it is possible to make satisfactory determinations of the monthly flow of all rivers. He does, however, believe it to be possible to make fairly accurate determinations of the monthly run-off from by far the greater portion of the water-sheds of the country, on the basis of reasonably complete physical data.

Table 48, showing run-off from a number of Wisconsin water-sheds during the winter, summarizes valuable data. The writer also holds in high regard Mr. Hoyt's discussion of the effects of ice on stream flow, as given in Water Supply Paper 337. Bi-monthly meter gaugings taken in connection with accurate temperature records, undoubtedly furnish a better basis for the estimation of winter discharge of streams than computations based on physical data alone. This fact does not prove, however, that there is no need for making estimates of winter stream flow on the basis of physical data where the available discharge records extend over only a few years. In fact, the necessity for such estimates would seem to be increased rather than diminished by Mr. Hoyt's own arguments. If the winter estimates of stream flow made previous to the publication of Water Supply Paper 337 are less accurate than those made since then, the need for extending and supplementing the records now being obtained would appear to be increased rather than diminished. It is so easy to lose sight of the fact that records of the past discharge of streams are primarily for the purpose of predicting what the probable yield of the same water-sheds will be in the future.

The writer has repeatedly emphasized the fact that his method of computing run-off is to be used primarily for the purpose of analyzing, supplementing, and extending observed stream-flow data. It is practically always possible for an engineer to obtain from a few months'

Mr. Meyer. to a few years' gaugings on a given stream before it is necessary to make any report whatever on the probable flow. On the basis of the available records of discharge and an intensive study of the given water-shed, reasonably accurate values for the evaporation and transpiration coefficients can be determined, and in this way an estimate can be made of the probable yield of the given water-shed, by the rainfall-loss method, which is far more reliable, in the writer's estimation, than a conclusion based merely on short-term records of discharge.

Mr. Hoyt states that the extreme maximum and minimum run-off cannot be determined at all accurately by the writer's method. That may be very true when maximum and minimum values of discharge in cubic feet per second, are viewed in the light of 25 or 50 years of accurate run-off records. On most streams, however, the available stream-flow data cover such a short period of years that the writer believes a more accurate estimate of flood flow can be made on the basis of physical data properly analyzed in connection with the available records of stream flow than the data afforded by the short-term records alone.

The writer does not believe, for example, that Mr. Hoyt would contend that the maximum flood flow on the Black River at Neillsville can be more accurately determined from the 4½ years of run-off records than from even the 9 years of computed precipitation minus loss, given in the paper for this stream. No run-off records are available for 1911, yet, according to Mr. Hoyt's figures, the computed precipitation minus loss for September was 3.67 in., and for October 7.27 in. The next highest computed precipitation minus loss for the summer or fall given in Mr. Hoyt's paper, is 4.14 in. for June, 1914, during which a total run-off of 4.11 in. was observed.

Though the writer has not had time to make a careful estimate of the probable maximum rate of run-off during October, 1911, it would appear from the large computed precipitation minus loss for September and October that the rate must have been very substantially larger than that which occurred in June, 1914. The daily precipitation records indicate that there was a precipitation on the Black River water-shed, above Neillsville, of about 1.8 in. on September 27th, and of about 0.5, 1.5, and 3.0 in. on October 1st, 3d, and 5th, respectively. Evidently, conditions were exceptionally favorable for an extreme flood. The rainfall between September 27th and October 3d, more than exhausted all the available soil storage capacity, so that most of the 3 in. of rain which fell on October 5th inevitably found its way into the stream, with the result that both The Dells and the Hatfield Dams failed, and a large portion of the business district of Black River Falls was swept away.

Although, as stated on page 631,* "The determination of the probable extremes of discharge for streams on which only short-term run-off

* *Proceedings*, Am. Soc. C. E., for March, 1915.

records are available is too large a problem to be discussed in the present paper", it may bear reiteration that such estimates must be made on the basis of the available records of stream flow, properly analyzed and supplemented by estimates based on rainfall and other physical data. It may here be remarked that, in making such estimates of probable flood flow, the writer has found it desirable to utilize isohyetal charts of excessive precipitation for the region in which the stream, for which the estimates of flood flow are desired, is situated. Such charts can be compared with the rainfall which produced the highest discharge during the period over which run-off records extend, as an aid in estimating the probable extreme flood. Mr. Meyer.

In discussing Mr. Hoyt's computations for the Black River water-shed, it may be well to call attention to the fact that the total computed precipitation minus loss from November, 1905, to April, 1906, inclusive, is only 5.00 in., whereas the observed run-off for the same period is given as 7.46 in. It requires no argument to show that this represents an impossible condition—either rainfall or run-off records being grossly incorrect.

No comparison can be made between annual computed precipitation minus loss and observed run-off for the Wisconsin River water-sheds, because about 45% of the drainage area above Rhinelander and about 25% of that above Merrill is under reservoir control. It will be noted, however, that the computed values of precipitation minus loss for the entire period agree very well with the observed run-off.

The annual computed precipitation minus loss for the Wisconsin River water-shed, between Rhinelander and Merrill, only about 5% of which is under reservoir control, agrees very well with the observed annual run-off, except during 1911. It is difficult to see, however, even while recognizing the sandy character of the Upper Wisconsin River water-shed, why the precipitation which occurred during July and August, 1911, did not produce more run-off than that given in Table 46. For example, the run-off decreased from 0.88 in. in June to 0.25 in. in July, and 0.39 in. in August, even though the precipitation increased from 1.51 in. in June to 7.05 in. in July, and 4.72 in. in August. If these figures are correct, and merely reflect the absorptive capacity of the Upper Wisconsin River water-sheds, surely no further argument is needed to prove that these water-sheds are indeed exceptional, and that the evaporation loss is necessarily very small.

The writer has not taken time to check Mr. Hoyt's figures, but it appears that in some cases the estimates of average precipitation for the different water-sheds will bear some revision. For preliminary computations, it may be well enough to average, arithmetically, and without weighting, the recorded precipitation for all stations on or near the given water-shed. Using the average of all available precipitation records on a given water-shed may also be satisfactory where the

Mr. Meyer. stations are numerous and well distributed. Where stations on a water-shed are few, however, and records of stations adjacent to the water-shed are used, they should not be given the same weight as those for stations situated centrally on the water-shed.

The writer intended to state in the paper that the rainfall and run-off records for the Ohio, the James, and the Roanoke Rivers were taken from Mr. J. C. Hoyt's paper, "Comparison Between Rainfall and Run-off in the Northeastern United States".*

Since the writer's paper was published, revised monthly run-off values for the Root River, at Houston, from March, 1913, to February, 1914, have been furnished by the Washington Office of the United States Geological Survey. These figures are as follows:

March	1.20 in.
April	0.46 "
May	0.39 "
June	0.30 "
July	0.43 "
August	0.30 "
September	0.22 "
October	0.25 "
November	0.22 "
December	0.23 "
January	0.24 "
February	0.17 "
Total	4.41 in.

Perhaps the most striking fact about these revised figures is that the run-off for March, 1913, as given by the Washington Office, is less than half that given by the District Office, as a preliminary figure. The difference in estimates no doubt arises mainly from the fact that gauge heights for the month are confined to the period from March 22d to 31st. The writer is inclined to accept his estimates, based on rainfall and other data, as being perhaps more accurate than either the estimate of the District Office or that of the Washington Office of the Geological Survey.

In conclusion, the writer desires to express his appreciation of the interest shown by those who have discussed the paper, and gratification at the general agreement with the fundamental principles underlying his method. He desires, further, to reiterate the often expressed sentiment that all methods of computing run-off from rainfall and other physical data should be used, primarily, for the purpose of analyzing, supplementing, and extending observed stream-flow records, so as to make these records a better basis for works of improvement into which run-off enters as a factor.

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* *Transactions,* Am. Soc. C. E., Vol. LIX, p. 431.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

TEMPERATURE CHANGES IN MASS CONCRETE

Discussion.*

By Messrs. Charles H. Paul and A. B. Mayhew.†

Charles H. Paul,‡ M. Am. Soc. C. E., and A. B. Mayhew,§ Assoc. M. Am. Soc. C. E. (by letter).—Since the preparation of the paper, nearly a year ago, additional data have been accumulated, none of which tends to contradict any of the original conclusions. Messrs. Paul and Mayhew.

The upper half of Plate XLIII gives the thermometer record for the whole year, 1914, amplifying the partial record for that year shown in the lower half of Plate VIII; and the lower part of Plate XLIII gives the record for 1915, as far as available. This latter brings out several interesting points.

Compare the records of Thermometers Nos. 7 and 8, the former being 2 ft. from an exposed face and the latter 10 ft. from the same face, and at the same elevation and cross-section. The record of Thermometer No. 7 shows the effect of the daily variations of outside temperature, and Thermometer No. 8, 8 ft. farther in from the face, gives a smooth curve, and shows very clearly the comparatively slight effect of daily variations of outside temperature. With a seasonal variation in mean daily temperatures of 73°, the seasonal variation of Thermometer No. 8, 10 ft. in from the face, was only 12 degrees. A careful comparison of these two records, together with that of the air temperatures, makes an interesting study. The hump in the curve of Thermometer No. 7, during the latter part of August, where the recorded temperature exceeds the mean daily temperature, is due to the fact that the maximum daily temperatures during that period were in the neighborhood of from 95 to 100°, and the warm part of

* Discussion on the paper by Charles H. Paul, M. Am. Soc. C. E., and A. B. Mayhew, Assoc. M. Am. Soc. C. E., continued from August, 1915, *Proceedings*.

† Authors' closure.

‡ Arrowrock, Idaho.

§ Dayton, Ohio.

Messrs. Paul and Mayhew. the day continued for more than its share of the time, so that a mean of hourly temperature readings would give a result considerably higher than the mean daily temperature as computed ordinarily.

Comparison of the records of Thermometers Nos. 7 and 9 is interesting in showing the performance of two thermometers at the same elevation, each close to an exposed face. Thermometer No. 7 is 2 ft. from the down-stream face (west exposure) and Thermometer No. 9 is 1 ft. from the up-stream face (east exposure). The water in the reservoir was above the elevation of Thermometer No. 9 from May 11th until August 24th, 1915.

The record of Thermometer No. 5, as compared with that of Thermometer No. 9, shows the effect of the water in the reservoir on the temperature of the concrete close to the up-stream face. The water was above the elevation of Thermometer No. 5 after February 26th, and the temperature of the water at that elevation was between 32° and about 52° during the remainder of the period shown on Plate XLIII. At the elevation of Thermometer No. 9 (covered from May 11th to August 24th), the water was considerably warmer, especially after the early part of August, when the surface was not far above that elevation.

The records for Thermometer No. 4 now extend over a period of nearly 2½ years. It was covered at first to a depth of 3½ ft. and left thus for several months during the summer of 1913. Late in October, 1913, concreting over this thermometer was resumed, and the effect of this was felt until about February 1st, 1914, when the temperature registered was 71 degrees. Since that time the temperature has been falling very slowly and steadily. It has reached 58° at the date of this writing (September 1st, 1915), and an examination of the curve during that period shows how very little the temperature at that distance from an exposed face is affected by the seasonal variations of air temperatures. There is just a slight gradual change in direction of the curve responding with a lag of several months to the extremes of seasonal temperatures.

The curves for Thermometers Nos. 1 and 4 are almost parallel (since February, 1914), as would be expected. A comparison here shows that the original temperature of the concrete makes its effect felt for a very long time. Ultimately, we would expect these two thermometers to register temperatures not far apart, with No. 4 slightly higher than No. 1. Assuming that No. 1 will finally register about 40°, which is approximately the mean temperature of the water at the bottom of the reservoir, and that it will continue to drop at the same rate as it has for the last 12 months, it will have taken 5 years after placing for it to reach its fixed temperature.

Thermometer No. 10 was placed on March 12th, 1915, in fresh concrete, and covered to a depth of 2 ft.; within 24 hours it was

PLATE XLIII.
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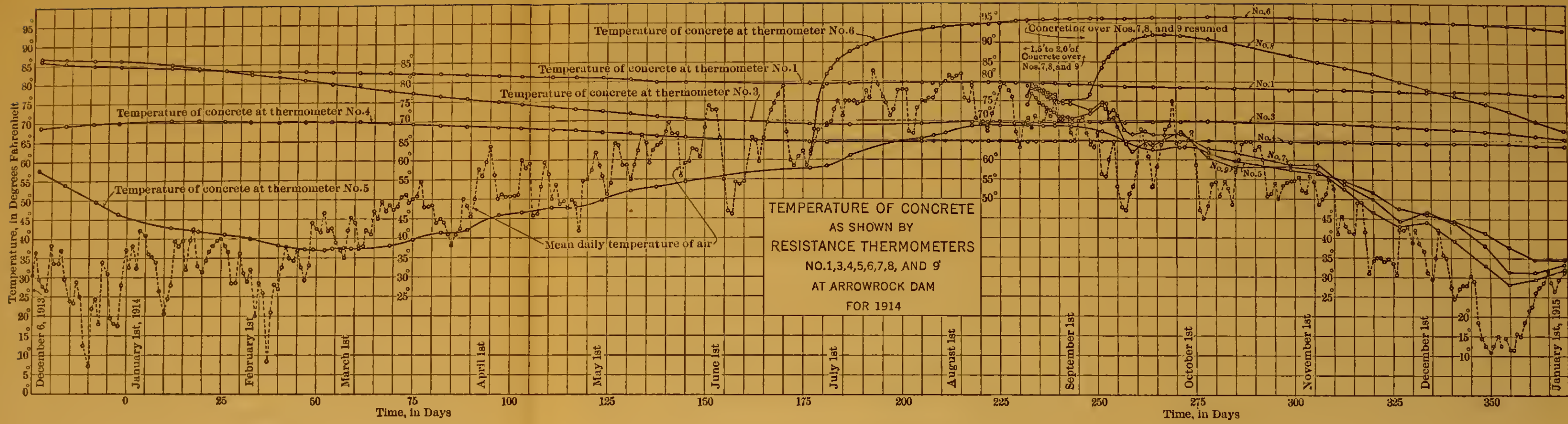
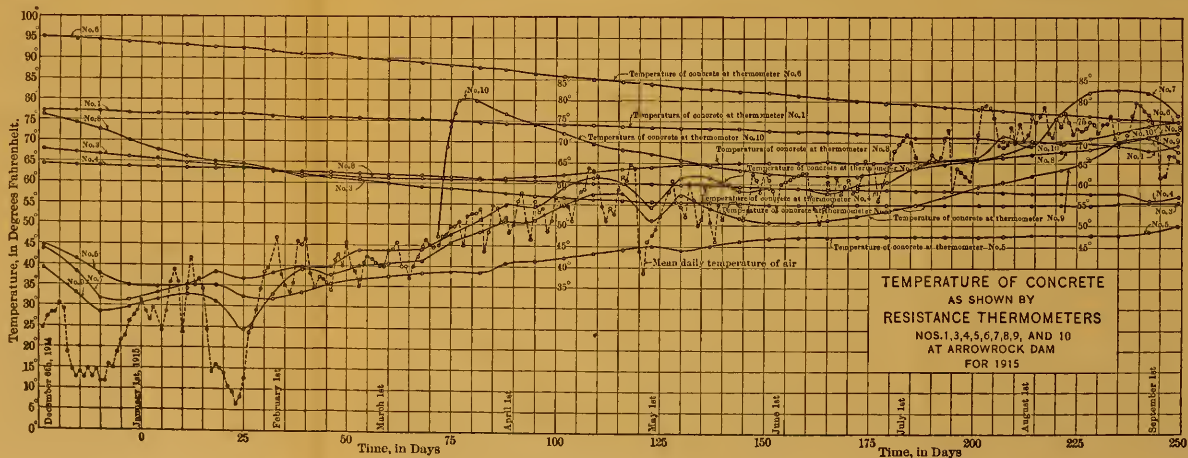
TEMPERATURE CHANGES IN
MASS CONCRETE.





Messrs. Paul and Mayhew.

covered to a total depth of about 6 ft. Its record shows the results that would be expected after a study of the others.

With records of nearly 2½ years for some of the thermometers, and after having the reservoir filled within 28 ft. of the top of the dam during the season of 1915, all the thermometers (with the exception of No. 2) continue to give readings that indicate perfect condition of the apparatus. This is very gratifying, in view of the difficulties experienced heretofore in obtaining satisfactory results for any length of time with similar apparatus.

For the benefit of those who may wish to go further into the effect of water in the reservoir, Table 1 gives the record of the elevations of water surface at various dates during the season of 1915.

TABLE 1.—ELEVATIONS OF WATER SURFACE IN ARROWROCK RESERVOIR.

Date.	Elev. Water Surface. Rising.	Date.	Elev. Water Surface. Falling.
February 15th	2 972	June 12th	3 183
March 1st	3 020	July 1st	3 171
" 15th	3 033	" 15th	3 155
April 1st	3 049	August 1st	3 136
" 15th	3 069	" 15th	3 118
May 1st	3 085	September 1st	3 086
" 15th	3 119		
June 1st	3 173		
" 12th	3 183		

The writers are pleased to know that investigations similar to those undertaken at Arrowrock are being carried on at Kensico. It is gratifying, though not surprising, that in all essential points the results of these two independent researches have led to similar conclusions.

Mr. Seabury misunderstands the intentions of the writers when he states: "At the Arrowrock Dam, an effort seems to have been made to reduce to a minimum the effect of the setting cement, * * *." The only object the writers had in mind was to obtain a record of temperature changes in mass concrete under average conditions. Some of the thermometers were covered a few inches and some as much as 3½ ft.—most of them from about 1½ to 2 ft.—when placed. Some were placed close to an exposed face, and some well back in the mass. The writers believe that this gives records very close to the average for the class of work represented by large dams, where it is not usual to place a layer of concrete more than from 2 to 4 ft. in thickness, at any one place, during one shift. Maximum possible temperatures are undoubtedly of great interest, but the writers believe that average temperatures —or perhaps average maxima for any considerable volume, like a day's

Messrs. Paul and Mayhew. work—are more valuable; for the practical use of these data is in their application to the design of dams or other large structures, where good-sized blocks may be considered as units, and where the use of maximum possible temperatures, obtained under conditions not corresponding to actual practice, might lead to unwarrantable results. It has been the writers' experience that concrete in large dams is usually placed in about 2-ft. layers, and that it is not usual to place more than two consecutive layers over any considerable area during one shift.

Keeping in mind the fact that the writers are considering average results for masses of several hundred yards spread in layers, and that Mr. Seabury, if understood correctly, is thinking of maximum possible temperatures in masses smaller in volume, or at least in superficial area, it is not difficult to bring into accord the opinions on any essential points. For instance, referring to Mr. Seabury's comments on Conclusion (1), radiation always does take place, to a large extent, under ordinary conditions of placing mass concrete, and very rich mixes are not used ordinarily in work of that class.

Sand-cement concrete sets more slowly than straight Portland-cement concrete, and probably, therefore, the maximum temperature of a mass of sand-cement concrete placed under ordinary conditions would be lower than that of Portland concrete similarly placed.

Mr. Seabury's record of the experiments with Alsen and Atlas cement concretes placed under similar conditions is very interesting. No mention is made as to which is the quicker-setting cement, but the writers would guess that the Atlas was a little the quicker, and that, under ordinary conditions of placing, it would show a slightly higher maximum than the Alsen.

Mr. Wilson Fitch Smith's observation *A* corresponds very closely with the results obtained at Arrowrock. His observation *B* suggests this comment, that during the time the temperature is increasing on account of chemical action the concrete is losing moisture, which tends to lessen the effect of the increasing temperature on its change of volume. The writers believe, however, that Mr. Smith has suggested a more or less serious objection to the too rapid placing of concrete in confined masses with comparatively small superficial areas. In general, as he states, his conclusions are similar to those of the writers.

Without being able to give results of experiments to support their opinions, the writers would answer Mr. Immediato's questions as follows:

1.—A quick-setting cement will probably give higher temperatures, both intermediate and final, than a slower-setting cement, assuming that the concrete is spread in layers over comparatively large areas, and that all conditions are similar.

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2.—The effect of free lime is not known.

3.—Generally speaking, the denser concrete would give higher values.

4.—Excess of water would probably tend to decrease the maximum temperatures.

Regarding Mr. Williams' first comment, it is impracticable, of course, to check the thermometers after they are once placed; but by comparing them with one another, it is possible to judge as to their accuracy. The records obtained at Arrowrock have been very satisfactory in that respect, and the troubles mentioned by Mr. Seabury have not been encountered there—always excepting Thermometer No. 2, which went bad soon after it was placed. It might be said, concerning this thermometer, that apparently it is slowly recovering, although as yet no records, except those shown on Plate VIII, are being used.

Mr. Walter M. Smith gives some very interesting data in regard to temperature changes in arches. As these sections are all comparatively thin, and the concrete is comparatively rich, the conditions are hardly comparable with those at Arrowrock, and different results, therefore, would be expected.

As to the quantity of Portland cement in the Arrowrock concrete compared with the ordinary 1:3:6 mix, the point should not be overlooked that in sand-cement the Portland content is very finely ground, and therefore more of it is active than in commercial Portland cement. It has been shown at the Arrowrock laboratory that about 30% of commercial Portland cement is retained on a 280-mesh sieve, and that this 30% is practically inactive when used as cement. Probably some that just passes this sieve is also inactive, but no apparatus was at hand to determine that; however, it is not unreasonable to assume that at least 35% of commercial Portland cement is inactive. This residue, when reground to pass a 200-mesh sieve, is more active than ordinary Portland cement. Of the sand-cement, as manufactured at Arrowrock, only 15% is held on a 280-mesh sieve, and of that residue practically all is blending material, so that almost all the Portland in sand-cement passes the 280-mesh sieve, and is finer ground and more active than the corresponding 70% of commercial Portland. The difference in active cement content between the 1:2½:5 sand-cement concrete used at Arrowrock, and ordinary 1:3:6 Portland cement concrete, therefore, is very small. The 2¾ parts of cobbles used at Arrowrock corresponds to the plum rock used commonly in mass work. It is true, however, that the paper deals with comparatively lean mixtures, as are common for mass work, and Mr. Smith's comment on that point is well founded.

Conclusions (2) and (3) were based on the results shown by Fig. 8, where, it will be noted, temperatures of air and concrete were taken every 2 hours for a 48-hour period, and after the effect of chemical

Messrs. Paul and Mayhew. action was fairly well overcome. Fig. 7, covering a period immediately after the placing of the thermometers, would not be expected to give reliable data on that particular point.

Mr. Wiley shows clearly the advantage of cold-weather work, which is beyond question. Unfortunately, however, it is not always possible to avoid concreting during the warmer months, and, in any case, the writers believe that contraction joints are desirable.

It is Mr. Wiley's discussion that opens up the question of contraction joints, which properly goes with the subject matter of this paper. Fig. 9 shows the design and spacing of contraction joints in the Arrowrock Dam.

By taking advantage of the inspection galleries, especially the one at Elevation 3 090.5, it has been possible to measure the movement of fifteen of these joints (spaced 50 ft. apart at that elevation), not only at the up-stream face of the dam, but also at the up-stream and down-stream walls of the inspection gallery, 16 ft. and 22 ft., respectively, in from the up-stream face of the dam. The average maximum opening of these fifteen joints was as follows:

At up-stream face......................	0.0708 in.
16 ft. in from up-stream face............	0.0444 "
22 " " " " "	0.0324 "

The average minimum opening of these fifteen joints, subsequent to the maximum opening, was:

At up-stream face......................	0.0064 in.
16 ft. in from up-stream face............	0.0300 "
22 " " " " "	0.0240 "

These records are for one season only, and undoubtedly the heat in the concrete, due to setting, was still being felt, but they are given for what they are worth. Later records along this line will be of interest and value. It is believed, however, that these records, meager as they are, indicate the value of contraction joints in structures of this kind.

The writers would add to the "Conclusions" given in the paper, the following:

(6) (Superseding No. 6 in paper.) The seasonal variation in the temperature of concrete 10 ft. from an exposed face is about 12°, when the seasonal variations of the mean daily temperature of the air is about 72 degrees.

(7) The seasonal variation in the temperature of concrete 20 ft. from an exposed face is very little, and after the effect of "setting heat" has once been overcome, the change in temperature of concrete at that distance from an exposed face is so slight as to be negligible, under ordinary conditions.

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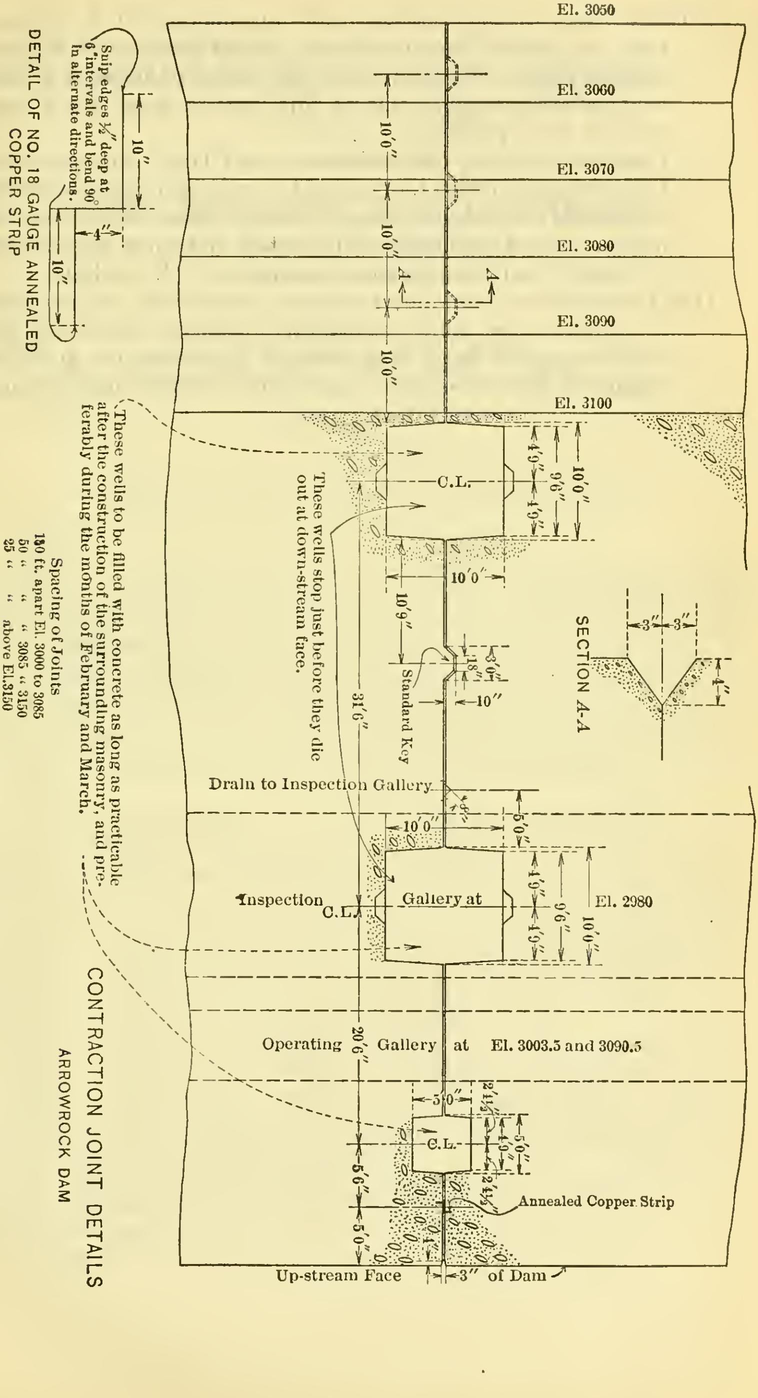

CONTRACTION JOINT DETAILS
ARROWROCK DAM

FIG. 9.

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(8) The effect of the "setting heat" in concrete 20 ft. or more from an exposed face is felt for several years after the concrete is placed. Concrete near the center of mass of a large dam probably retains some of this "setting heat" for 5 years or more after placing.

(9) Contraction joints are desirable in all large concrete structures, whether or not they are to be exposed to wide variations of outside temperature, except possibly when all construction may be carried on during cold weather, and when the variation of outside temperatures, after completion, will be slight.

(10) Changes in volume due to setting, hardening, and seasoning of concrete, are more important, ordinarily, excepting very close to exposed faces, than changes in volume due to the influence of daily or seasonal variations in outside temperatures.

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PAPERS AND DISCUSSIONS

THE PUMPING PLANT OF THE MORENCI WATER COMPANY

Discussion.*

By Messrs. H. Hawgood and W. L. Du Moulin.†

H. Hawgood,‡ M. Am. Soc. C. E. (by letter).—The Morenci pumping plant is an instance of special construction to meet special conditions in an extremely rough and precipitous country. The author and his associates are to be congratulated on their successful solution of the inherent difficulties of the situation. He would add to the completeness of his valuable paper by giving the ordinary revolutions per minute of the pumping engines, and also the quantity of water pumped through the 10-in. line with a friction head of 175 ft., as stated. Mr. Hawgood.

The writer has visited the plant on two or three occasions. The pumps run smoothly, without jar, notwithstanding the absence of air chambers. When the difficulties of charging and re-charging air chambers with air at about 800 lb. per sq. in. are considered, their omission is sound. It is probable, however, that their introduction at points of moderate pressure on the pipe line would mitigate the water-hammer caused by cyclic variation in rate of flow, due to uncushioned contact with crank-driven plungers.

The water-hammer in the last 5 000 ft. of the line was rhythmic and heavy. The time interval between blows, however, did not appear to be uniform or entirely in unison with the pump speed. This may have been due to the numerous intervening summits on the pipe lines affording opportunity for air pockets.

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* Discussion of the paper by W. L. Du Moulin, Assoc. M. Am. Soc. C. E., continued from September, 1915, *Proceedings*.

† Author's closure.

‡ Los Angeles, Cal.

Mr. Hawgood. The relief given by the inward-opening check-valves, described by Mr. Du Moulin, proves the correctness of his diagnosis as to the cause of the local water-hammer.

It is suggested that free discharge into small open tanks at suitable elevation at the highest point on the line, and steady gravity flow from there to the storage tanks would eliminate the hammer troubles and obviate the flow disturbance which must be created by the introduction of air. Proper balance between the outflowing gravity water and the inflowing pump water could be maintained by one of the several simple devices for that purpose. Air chambers at the critical high points in the 5 000-ft. stretch might accomplish the same results.

The settling system is particularly interesting. The problem of creating along the sides of a box canyon a system sufficient to clarify the storm-waters has been worked out in a very practical manner. The material increase made in the available supply, and the increase in efficiency and life of the pumps obtained by the introduction of the settling plant, certainly warranted its cost.

The filtration through the river gravels of the supply destined for domestic use has its counterpart in the growing practice in Southern California of spreading the excess winter stream flows over the gravel débris cones at the canyon mouths. The underground storage thus accomplished is available later for summer pumping. The average infiltration rate is about 2 sec-ft. per acre. Gravelly lands covered with brush are generally more absorptive than the bare gravel and boulder beds of the streams themselves. This phenomenon is due to sealing by silt. Violent floods tear up the beds and renovate the absorptive capacity. It would be interesting to hear from Mr. Du Moulin as to the methods he has adopted to obtain new infiltration surfaces.

Mr. Du Moulin. W. L. Du Moulin,* Assoc. M. Am. Soc. C. E. (by letter).—The duty test of the cross-compound pumping engine was made within 4 months after that engine was started, and was in the nature of an acceptance test, in which the conditions were as similar as practicable to those called for by the specifications.

The duty test of the triple-expansion pumping engine, however, was made after it had been run several years, and was more for the purpose of determining the duty that it might develop under favorable test conditions. For this the Water Company employed a responsible testing engineer. Everything that was essential was done in order to gain accurate results, although possibly some slight refinements were disregarded, as it was not an acceptance test; but, neglecting to make such refinements did not, in the writer's opinion, affect the results essentially. The data show what was accomplished under the conditions indicated. As the auxiliaries are of the attached type, the steam required by them during the test was naturally included in that

* Morenci, Ariz.

charged to the engine. No correction was made for the 42° Fahr. higher superheat obtained than was called for under the guaranty. With this correction, the duty would have been in the neighborhood of 177 808 000 ft-lb. No allowance was made for pump slippage. The duty was based on plunger displacement. Tests on these pumps have shown the slippage to be between 1½ and 2 per cent. Naturally, the pump was carefully examined before the test, and all packing and valves were put in first-class condition, with not the least suspicion of a leaky valve in the pump. Under such conditions, it is generally customary not to take the pump slippage into account. Moreover, the quantity of water leaking past the plungers was so small that it was of no practical consequence. This small quantity goes out under the working pressure, and represents work just as though it escaped through the discharge pipes. The plungers, moving against water pressure, do work for the full length of their stroke. With a reciprocating pump in good condition, as was the case in each instance during the Morenci tests, the duty, based on plunger displacement, is as accurate and satisfactory for all practical purposes as though slippage had been taken into account, especially when weir measurements and the Venturi meter usually do not check each other within 2 per cent. In the writer's opinion, duty based on plunger displacement is really the only reliable means of comparing the performance of large reciprocating pumping engines built by different manufacturers. It reduces the matter of personal equation very nearly to the minimum. Of course, in comparing the duty of a reciprocating with a centrifugal pumping engine, the slippage of the former should be ascertained and an allowance made, as the duty of the centrifugal pump is usually measured in foot-pounds in the water. However, in such case, the error would not be great enough to affect the result materially if slippage were not taken into account. Mr. Du Moulin.

In conducting a duty test, the object is either to determine whether a pumping engine comes up to its guaranty, or to find out what duty it is capable of developing under favorable conditions. This latter information is particularly of interest from an engineering standpoint. In the tests of the Morenci engines, the aim was to obtain fair results; and, as stated, no attempt at "boosting" conditions, and consequently results, was made.

A purchaser generally knows what his operating conditions will be, and is able, therefore, to give the necessary information, showing under what conditions the pumping engine is to work, with a fair degree of accuracy. Sometimes a bad "guess" is made, however, or again it frequently happens that unforeseen circumstances compel changes after a pumping engine has been ordered, so that the actual operating conditions vary materially from those for which it was designed. As a consequence, the average duty developed by the engine under operating

Mr. Du Moulin.

conditions will fall far below that obtained under test conditions. The writer is of the opinion that this fact should not count against the engine or its designer; for, if the average conditions were more favorable, a better everyday duty could be undoubtedly maintained. A bad "guess" or altered or unfavorable operating conditions may account to some extent for the very large discrepancy in the case cited by Mr. Potter, between the everyday performance and the duty test, as well as the fact that the result of the test was "boosted" by methods which were far from equitable. In the Morenci case, the actual operating conditions varied from those anticipated when the triples were ordered. The cross-compound was put in later, and the builders were able to design this engine to conform more closely to the everyday conditions. This, together with the better vacuum that this engine carries, accounts perhaps for the fact that its everyday performance is not only equal to, but slightly better than, that of the triples. There is no doubt that, with a better vacuum and more favorable operating conditions, the performance of the triples would be the better.

For 1912, the actual water pumped was 94.5% of the plunger displacement. The improvement in the efficiency of the pump end has been gradual, and has been the result of changes mentioned in the paper, and of detailed attention given this matter during a period of some 5 years. The report on the property of The Morenci Water Company, made by Mr. C. E. Sloan, Engineer for the Arizona Corporation Commission, in 1913, contains the records to that date. Mr. Sloan and his assistants made a careful examination of the property and the records, and his report substantiates statements made by the writer in regard to pump end performances. The improvement in the performance of the pump end of the Morenci pumps has been about 18 to 20% since they were started. The maintaining of this improved performance is due principally to the detailed attention given to plunger packing and valves. It is the aim to maintain them continually in first-class condition. The pressure is very great, and leaky valves soon do much damage. As soon as a leaky valve is noticed, it is immediately replaced by a good one. This is not the general practice in pumping plants. In fact, the care of valves is a feature that is neglected. In most plants, the practice seems to be of an arbitrary nature, the valves being overhauled once every year or two. The writer has visited a great many pumping plants of all sizes, and it is his opinion that in most of them a very large saving can be effected by giving more detailed attention to the valves, thereby reducing excessive slippage and leakage, and the loss represented thereby. In one instance, the slippage was more than 30 per cent. Overhauling valves more frequently will fully pay for itself in the improved performance of the plant.

Mr. Du Moulin.

Mr. Duncan's description of the filtration gallery that he constructed to obtain additional clear water is very interesting, and, under the right conditions, is a very feasible method, but it would not be practicable at Morenci. The formation of the bed of the Eagle River though similar, differs from that of Duck Creek at McGill, Nev. It consists of a river gravel but a great part of it contains more silt and there are also layers of silt running through the bed. These layers form quicksand when saturated with water, and are practically impervious when relatively dry. A hole was sunk to a depth of 21 ft. in the river gravel on the bank of the stream less than 8 ft. from the running water on the surface. The gravel was fairly moist at that depth, but no water in quantity collected in the bottom, thus plainly demonstrating to what extent the layers of silt seem to retard the percolation of the surface water. A tunnel was started across the river bed at a depth of about 40 ft. After proceeding some distance, it caved in, breaking up the ground to the surface and causing quite a depression. However, the silt sealed up the broken ground so completely that the water from the surface was prevented from filtering through in quantity. As the silt seals up the river bottom and sump hole sides and bottom so quickly during the rainy seasons, the method described by Mr. Duncan would be impracticable at Morenci. During the rainy season, the river is generally almost continuously muddy, and brings down the exceptionally large quantities of heavy, thick silt which settle quickly and do the damage. The writer's experience has demonstrated that, in general, the conditions at Morenci during the rainy season are not favorable for any kind of filtration scheme, unless the silt is first allowed to settle out of the water to a great extent by passing it through settling basins. As the settling system water is suitable for mill purposes, and there is sufficient well water for all other purposes, it was not necessary to incur the additional expense of a filtering system. Sufficient work was done toward the development of additional well water to demonstrate that it was practicable. The cost, however, would be very great, because of the isolated location and the expense of getting the necessary supplies to the plant. The settling system was the most practical and the least costly way of obtaining a satisfactory water. It had the additional advantage of eliminating the cost of lifting water out of a deep well or sump, as the settling system water flows by gravity to the pumps. Also, its cost of maintenance is very low, in fact inconsequential.

Mr. Hawgood's description of the method of obtaining underground storage in Southern California is very interesting. The observations of the writer bear out Mr. Hawgood's remarks on the effect of violent floods in increasing the absorptive capacity of the river bed. There have been several such floods on the Eagle River during the past 6 years, caused by rains and melting snows in the mountains, with the

Mr. Du Moulin. result, in each instance, that the yield of the well was materially increased during the succeeding months, and until the next rainy season. As soon as that season arrived, the small floods and muddy water sealed up the river bottom, with a consequent decrease in the well supply, and, in order to maintain the well at a certain capacity, infiltration surfaces had to be used. For reasons given in the paper, it was impracticable to use this method during the rainy season to obtain a sufficiently large quantity of clear water for all needs. Before the construction of the settling system, the infiltration surfaces were limited to a series of shallow pits that were cleaned very frequently with pick and shovel. This was tedious and rather expensive. Since constructing the settling system, the area of infiltration surface is limited to the single large pit mentioned in connection with the auxiliary settling basin. Because of the better water, this surface need only be renewed once or, at the most, twice a year. This is done by washing the silt down the sides of the pit with high-pressure water, which is available, and pumping out the thick muddy water from the bottom with several bilge pumps.

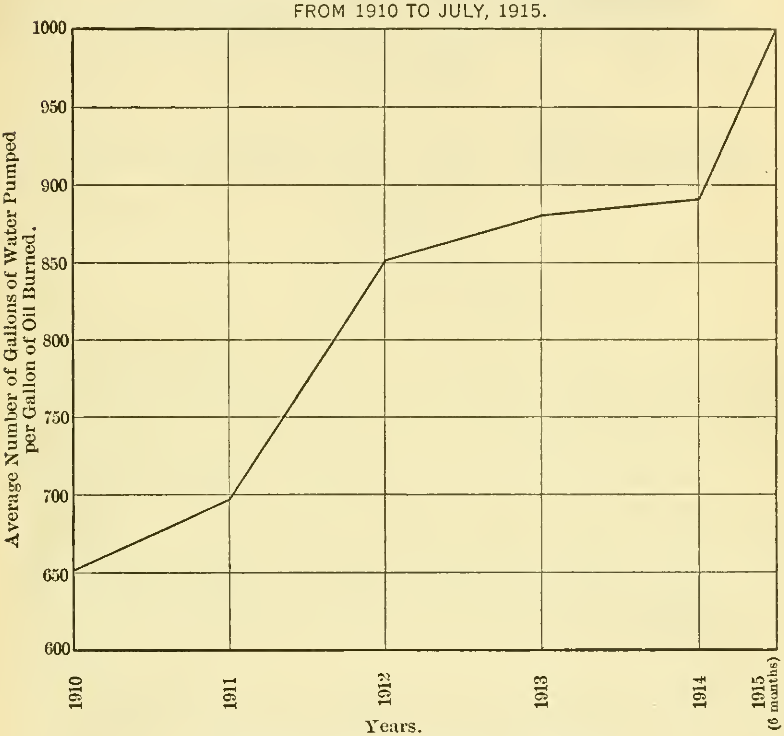
Careful inspection of all exhaust piping and exhaust connections has been made frequently, and no air leaks have been found of sufficient consequence to account for the low vacuum. All engines are equipped with recording vacuum gauges, and any variation in the vacuum is investigated. Since the paper was written, a jacket of galvanized iron has been placed around the condenser shell, with a space of 1½ in. between the latter and the jacket. Through this jacket, around the outside of the condenser, water is circulated, the space being kept full of circulating water. This arrangement caused an improvement of 3 in. in the vacuum, showing that the area of effective cooling surface is insufficient. The fault lies principally in the design of the condenser. The vacuum was measured at a point between the oil separator and the condenser inlet, close to the latter, a mercury column being used. All boilers are equipped with surface blow-offs.

The writer notes with interest Mr. Duncan's remarks in regard to the fact that his experience indicates the expectation of 30% increase in steam consumption in a plant of high-grade Corliss engines 10 years old. In a power plant with high-speed machinery, the writer believes that this might be the result; but low-speed pumping machinery, with the uniform load and service resulting from pumping into a reservoir, should not show with age much variation from its original steam economy, if properly maintained. One of the Morenci triple-expansion pumping engines has been in service almost continually for 6 years; and its performance to-day is, if anything, better than it was originally.

The curve and data on Fig. 29 give the history of the plant performance during the past 5 years. The improvement from the end of

1910 to July, 1915, has been 52%, and the average number of gallons of water actually pumped at present is about 1 000 for each gallon of oil burned. This is not based on displacement, but on actual water pumped. This improvement was effected through means described in the paper. Mr. Du Moulin.

CURVE OF PLANT PERFORMANCE.
AVERAGE NUMBER OF GALLONS OF WATER PUMPED
PER GALLON OF OIL BURNED, FOR EACH YEAR
FROM 1910 TO JULY, 1915.



Average for 1910................656 gal.
Average for 6 Months of 1915.......998 gal.
Improvement in Plant Performanee
from 1910 to July, 1915............52 per cent.

FIG. 29.

All water is pumped into storage tanks at Morenci, and is measured by Venturi meters as it leaves them. The quantity of water from the high-pressure lines used at the plant for various purposes is ascertained by measurements and tests, and in this way a comparatively accurate record is obtained of the actual quantity pumped.

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PAPERS AND DISCUSSIONS

WATER SUPPLY OF THE SAN FRANCISCO-OAKLAND METROPOLITAN DISTRICT

Discussion.*

By C. E. Sloan, Esq.

Mr. Sloan.

C. E. Sloan,† Esq. (by letter).—Notwithstanding the author's full though concise presentation of this subject, and the interesting discussions which the paper has brought forth, several obvious and important questions are suggested, but are not answered. Some of these have been hinted at in the discussions; others have not.

The title of the paper suggests the first, and perhaps the most important, of these questions, so far as the people of San Francisco are concerned, that is, if this so-called Hetch Hetchy water supply is a water supply for the San Francisco-Oakland Metropolitan District, why is it that San Francisco is acting alone and bearing all the burden?

If it is to be a water supply for the Metropolitan District, why has not that District been formed, and why should not the $45 000 000 in bonds, voted some years ago by San Francisco for a water supply, justly rest on all the Metropolitan District? At none of the many stormy sessions held within the past 10 years, during which Hetch Hetchy has been argued *pro* and *con,* has any real effort been made to establish a Metropolitan District. Why? Principally because the men behind the Hetch Hetchy project are pretty well satisfied that an election to form such a district would be overwhelmingly defeated in the East Bay cities, unless San Francisco would not be able to control the District, in which latter event the Hetch Hetchy project might very well be abandoned.

* Discussion of the paper by H. T. Cory, M. Am. Soc. C. E., continued from September, 1915, *Proceedings.*

† San Francisco, Cal.

Mr. Sloan.

Then why the proposal for a supply sufficient for all the Bay cities? The Grunsky project in 1902 was for only 60 000 000 gal. daily. The answer is this:

At the time of revoking the "Garfield Permit" by the Secretary of the Interior, the City of San Francisco was instructed, among other things, to investigate all other sources of supply available, and to lay before the Department of the Interior the results of this investigation. An ample supply for San Francisco alone would be 200 000 000 gal. daily, approximately five times the consumption in 1915. From the report of the Board of Army Engineers it will be seen that there are several sources which will furnish such a quantity, and the report clearly points out the fact that the needs of San Francisco can be supplied for many years to come without any water from outside sources. Numerous engineers have reported on the resources of the Spring Valley Water Company, and almost all agree with the Army Board that, by developing these sources to their ultimate capacity, San Francisco will not require water from an outside source until 1965. The report of the Board of Army Engineers, excluding half of the possible development of the Peninsula coast streams, places this date at 1947. This being true, it is obvious that a supply for San Francisco proper, from outside sources, of twice 200 000 000 gal., or 400 000 000 gal. daily, is not only a very expensive luxury, but a foolish undertaking for many decades to come.

As stated in the Army Board reports, there are several excellent sources, good for 200 000 000 gal. daily. There are only three good for 400 000 000 gal. daily—the Sacramento River, the McCloud River, and the Hetch Hetchy. Nothing simpler: all but these three were at one stroke absolutely eliminated by raising the requirements to 400 000 000 gal. daily. To justify this large demand, only the Metropolitan Water District was needed and suggested—but not formed.

San Francisco is particularly fortunate, compared with the remainder of the prospective Metropolitan District. A careful study of the population curves, water consumption, and immediately available supplies of the Metropolitan District, shows that San Francisco can do without a mountain supply for 50 years, but such is not the case with the East Bay region and Marin County; these must look to some outside source for their water in the very near future. It would seem, therefore, that these communities might well pool their needs and join in a solution of their problems.

The various sources of supply suggested in Mr. Cory's paper are considered with reference to a supply of 400 000 000 gal. daily. Just why he should have set the same minimum quantity as the proponents of the Hetch Hetchy project before Secretary Fisher he does not explain. His careful study of the question is convincing that this

quantity of water is unnecessary for the wants of San Francisco for possibly all time to come. Mr. Sloan.

Assuming, however, that 400 000 000 gal. daily is the ultimate demand of the Metropolitan District, the author finds that San Francisco would require only about 100 000 000 gal. daily in, say, 1960, and 200 000 000 gal. daily about 2000 A. D., the remainder being wanted by the rest of the Metropolitan District. If this fact were more generally known, the agitation which has been carried on for the past 10 or 12 years regarding Hetch Hetchy would practically cease, so far as San Francisco is concerned. As no effort has been made by the rest of the District to obtain an outside supply, it should be realized locally that San Francisco is ill-advised in planning to bring in a supply of which she can never hope to use more than one-half; for which (on a basis of assessed value) she will be compelled to pay about 70% of the total cost, and very probably all the costs; and the water of which she can only sell to other municipalities, under the terms of the Raker Bill, at actual cost.

However, it does seem that an outside source of water must be obtained for a Metropolitan District (exclusive of San Francisco), and that shortly. Without the financial backing of San Francisco, the Metropolitan District could not assume an obligation of bonds sufficient to obtain water from Hetch Hetchy. From the report of the Board of Army Engineers it will be seen that the quantity of water required for the East Bay region and Marin County is a great deal less than 400 000 000 gal. daily—indeed, about 200 000 000 gal. daily at most—and that this can be had from several different sources at considerably less cost than from Hetch Hetchy. Any of these sources could be financed by the remainder of the Metropolitan District.

Notable among these, as pointed out by C. J. Rhodin, M. Am. Soc. C. E., in his discussion,* is the Eel River supply.† Complete cost estimates on this source are not available; two reports, however, have been made public: one by Messrs. W. R. and N. A. Eckart to the Snow Mountain Water and Power Company, and one by Mr. George S. Nickerson to the President of the Modesto Irrigation District Water Users' Association. Mr. Nickerson's report is on a combination of the Southern Eel River-Putah Creek water-sheds, thereby furnishing a supply of 400 000 000 gal. daily. This would necessitate numerous expensive tunnels and the mixing of the waters of Eel River with those of Clear Lake, which latter waters, as stated in the report of the Board of Army Engineers, is not excellent. The water-shed of the South Fork of the Eel River is in Lake and Mendocino Counties, and almost wholly within the Stony Creek National Forest Reserve.

* *Proceedings,* Am. Soc. C. E., for May, 1915.

† See also Report of the Advisory Board of Army Engineers, p. 83.

Mr. Sloan. The territory tributary to Gravelly Valley Reservoir contains 268 sq. miles, and that tributary to Cape Horn Reservoir is 58.5 sq. miles, a total of 326.5 sq. miles. The rainfall in this territory is abundant:

Mean seasonal	43.51 in.
Maximum seasonal	78.62 "
Minimum seasonal	24.99 "

The ratio of maximum to mean rainfall, therefore, is 162% and of minimum to mean rainfall, 51½ per cent.

The seasonal mean run-off is 26.81 in., and the minimum run-off is 6.90 in. Thus, the ratio of mean run-off to mean rainfall is 55% and of minimum run-off to minimum rainfall is 27.6 per cent.

The Gravelly Valley Reservoir site is an excellent one. With a dam 150 ft. high, this reservoir would impound 70 000 000 000 gal., or 15.03 in., over the tributary water-shed. This would give an available water supply (during the dryest known period in the past 38 years) of 190 000 000 gal. daily, which could readily be increased to 200 000 000 gal. daily, by taking advantage of the available large storage reservoirs in the East Bay and Marin County regions.

The elevation of the intake of this system would be approximately 1 000 ft. above sea level, and, by following the proximity of the right of way of the Northwestern Pacific Railway Company, a gravity system could be constructed. The hydraulic gradient of this system would permit of the use of light-weight steel pipe or reinforced concrete conduit.

By crossing San Francisco Bay at or near McNear's Point, which crossing offers no great difficulties, the cost of the system would be obviously low. Furthermore, the waters of this water-shed are not now, nor in the future will they ever be, required for irrigation purposes.

The joining of the East Bay regions and Marin County in some such project, and keeping quite apart from San Francisco proper, is perhaps quite as possible as the carrying out of the Hetch Hetchy project along the present proposed lines.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE TWELFTH STREET TRAFFICWAY VIADUCT, KANSAS CITY, MISSOURI

Discussion.*

By E. A. Slettum, Esq.

E. A. Slettum,† Esq. (by letter).—*The Application of the Theory of the Continuous Beam in Calculating the Stresses in Frames with Rigid Connections.*—In connection with the investigation of the bents and columns for the Twelfth Street Trafficway Viaduct, as given in Mr. Howard's paper, it may be of interest to consider the application of the theory of the continuous beam in calculating stresses in any frame with rigid connections. A frame with rigid connections is one in which all members are connected at their intersection points in such a way that the angle between any two members meeting at a point will remain constant under all conditions of loading. A change in the direction of the end tangent of one member, therefore, will cause the same change in the direction of the end tangents of all other members meeting at the point. As this is the basis of the theory of the ordinary continuous beam, it is self-evident that this theory is applicable to such frameworks as described above. Mr. Slettum.

To illustrate the general application of the theory of the continuous beam for calculating the stresses in such frames, take the framing bent of a four-story structure, as shown in Fig. 29.

The moments of inertia have been assumed to be different for different members, but constant throughout the length of each separate member. The only difference in the case of a variable moment of inertia is that the coefficients in the three-moment equation change as shown under the theory of the continuous beam.

The frame will be assumed to be loaded in any manner, horizontally or vertically, by forces acting in its plane.

The influence of axial stresses on the deformation of the frame will be neglected.

* Discussion of the paper by E. E. Howard, M. Am. Soc. C. E., continued from September, 1915, *Proceedings*.

† Christiania, Norway.

Mr. Slettum.

$H_I + H_{II} + H_{III} + H_{IV} + H_V$ is the resultant of all horizontal forces, including wind load, concentrated at the different floor levels.

Under the application of external forces, the frame will deflect from its original position. The direction in which it will deflect cannot always be told beforehand. It will be convenient to assume, therefore, that the frame always deflects in a certain fixed positive

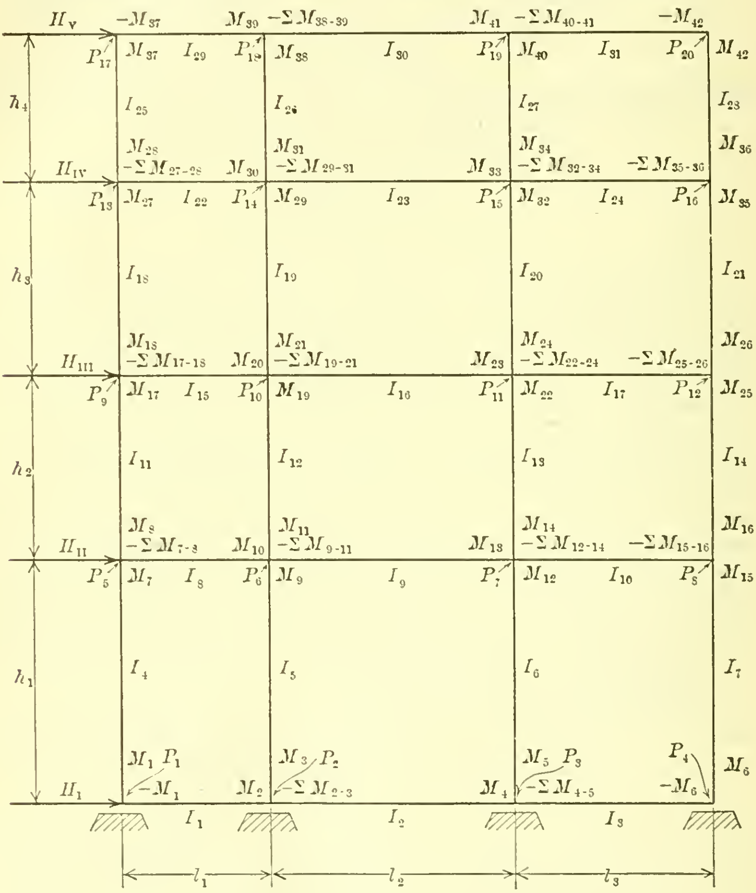


FIG. 29.

direction. By the sign of the figure giving its value we can then tell the actual direction. If it is plus, the frame deflects in the direction assumed; if it is minus, it deflects in the opposite direction. This also applies to the unknown continuity moments.

Assume that from left to right is the positive direction for deflections.

Mr. Slettum.

Assume a clockwise direction as positive for moments when applied to the panel points, and counter clockwise, when applied to the members.

In Fig. 30 is shown the deflected frame with the moments applied to the panel points. As each panel point is in equilibrium, the algebraic sum of the moments about any point must equal zero; hence, if all moments but one about a point are assumed to act in a positive direction, this latter moment will equal the sum of the others, and will be negative, as shown in Fig. 30.

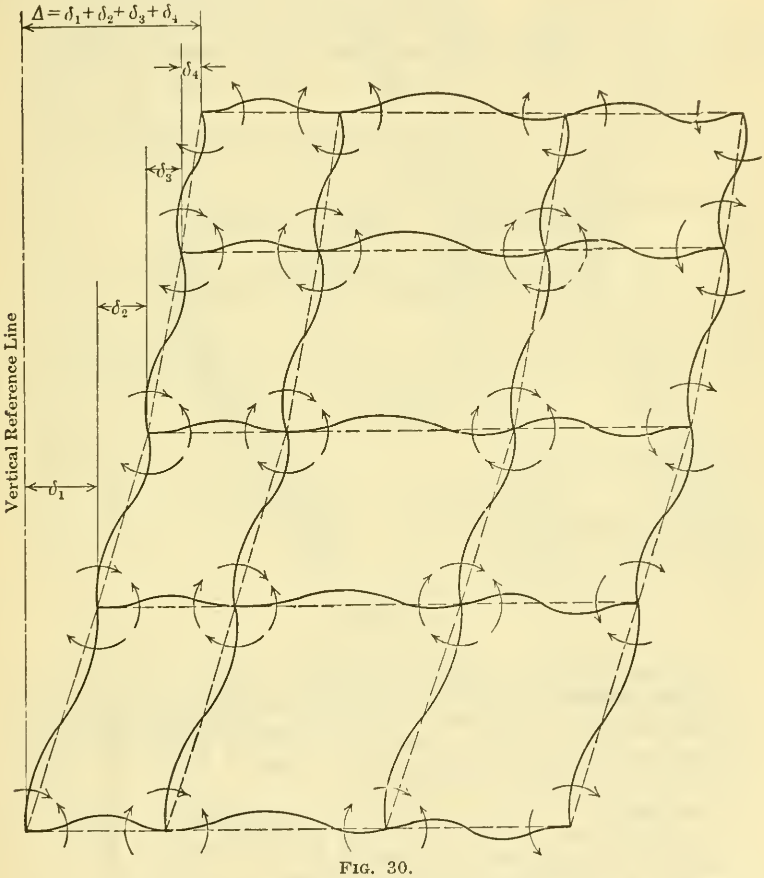


Fig. 30.

When speaking about beams, it is the usual practice to term moments producing tension on the bottom side "positive", and those producing tension on the top side "negative", the reason being that they go into the three-moment equations with positive and negative signs, respectively.

Mr. Slettum.

As far as direction goes, a so-called "positive" moment is positive on the left-hand side of a support and negative on the right-hand side, which is easily seen from Fig. 32. This double meaning of "positive" or "negative", therefore, is highly confusing.

The difficulty may be overcome by abandoning the terms "positive" and "negative" when referring to that kind of stresses produced on a certain side of the beam, and adopting, instead, the terms positive and negative side of the beam.

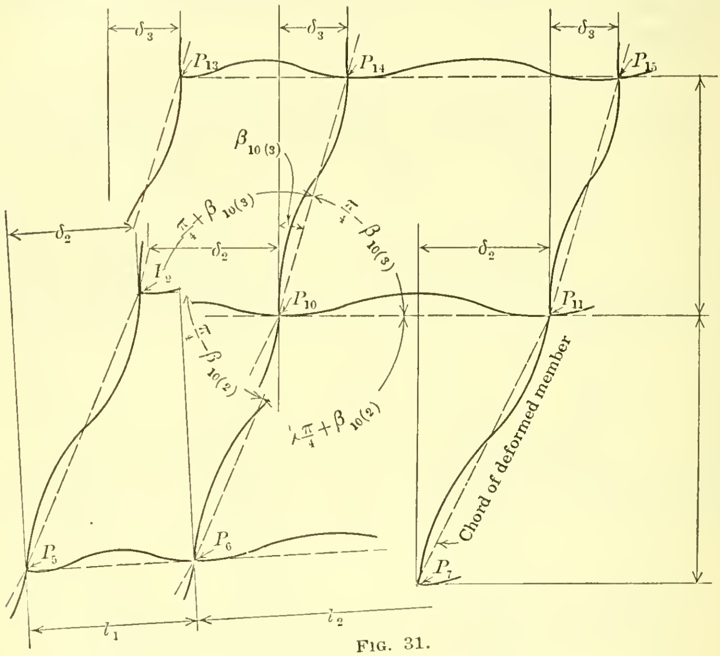
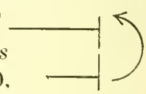


FIG. 31.

Having adopted a certain side of a bean moments, whether they have a positive or n itive, we know that the three-moment equation with positive sign direction, go into on the positive side of the beam, and with nega oducing tension sign when producing tension on the negative si of the beam.



Application of the Theory of the Continuous Beam to a Single Point.—Consider Panel Point 10, shown to a larger scale in Fig. 31. The deformations of the members shown in Figs. 30 and 31 actual deformations, but are those corresponding to the assu he tion of the continuity moments.

Mr. Slettum.

A three-moment equation can be written for any two members meeting at a point, but, as will be shown later, only as many of these are independent of each other as there are members meeting at the point, less one. Thus, if there are four members meeting at a point, we can write only 4 — 1 = 3 independent equations.

For convenience, therefore, we shall confine ourselves to members forming either a lower left, upper right, or lower right, corner, as shown in Fig. 33.

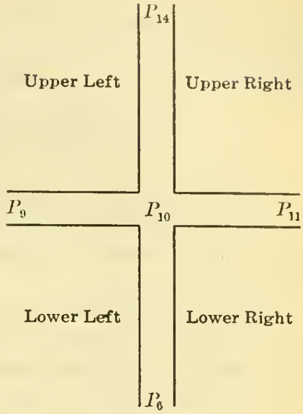


Fig. 33.

The three-moment equation for the ordinary continuous beam is:

$$M_0 \frac{C_0}{I_0} + 2\ M_1 \left(\frac{l_0}{I_0} + \frac{l_1}{I_1}\right) + M_2\ \frac{l_1}{I_1}$$

$$= C_0 + C_1 + 6\ E\left(\frac{\delta_1 - \delta_0}{l_0} + \frac{\delta_1 - \delta_2}{l_1}\right) \dots\dots\dots\dots (1)$$

The subscript, 0, refers to the left-hand span, and the subscript 1, to the right-hand span. (See Fig. 34.)

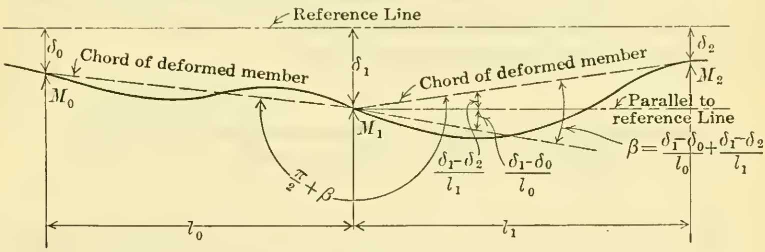


Fig. 34.

C_0 and C_1 represent the influence of the external loading.

For concentrations we have:

$$C = -\ \Sigma\ P\ \frac{l}{I}\ k\, l\ (1 - k^2).$$

For uniform loads:

$$C = -\ \frac{1}{4}\ p\ l^2\ \frac{l}{I}.$$

For external moments (or pairs of forces):

$$C = -\ \Sigma\ m\ \frac{l}{I}\ (3\ k^2 - 1).$$

Mr. Slettum.

By adding the subscript, 0 or 1, in the foregoing formulas, we get C_0 or C_1, as the case may be. $k\,l$ is the distance from the left-hand support, in the left-hand span, and from the right-hand support, in the right-hand span, to the point of application of the load.

$I =$ moment of inertia of the beam.
$E =$ modulus of elasticity of the material.

The other notations will appear from Fig. 34.

The external moments, m, go into the equation with positive sign when acting counter clockwise in the left-hand span and clockwise in the right-hand span.

The unknown continuity moments, M_0, M_1, and M_2, are assumed to act in such a direction that they produce tension on the positive side of the beam, which is the under side.

From Fig. 34 it is seen that the angle, β, which represents the change in angle between the two members, equals:

$$\beta = \frac{\delta_1 - \delta_0}{l_0} + \frac{\delta_1 - \delta_2}{l_1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Therefore, the three-moment equation (1) can be written:

$$M_0 \frac{l_0}{I_0} + 2\,M_1 \left(\frac{l_0}{I_0} + \frac{l_1}{I_1}\right) + M_2 \frac{l_1}{I_1} = C_0 + C_1 + 6\,E\,\beta \ldots\ldots (3)$$

Now, a frame differs from an ordinary continuous beam therein, that, instead of having the same moment, M_1, on both sides of the center support, it has different moments on the two sides, due to the action of the other members meeting at the point.

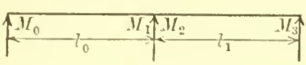


FIG. 35.

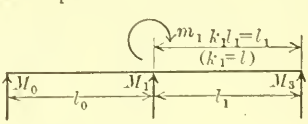


FIG. 36.

Let the moment on the two sides of the center support be M_1 and M_2, as shown in Fig. 35. M_2 may be considered as the resultant of M_1 and an external moment, m_1, acting at the point, as shown in Fig. 36. For this case we have:

$$M_0 \frac{l_0}{I_0} + 2\,M_1 \left(\frac{l_0}{I_0} + \frac{l_1}{I_1}\right) + M_3 \frac{l_1}{I_1} = -\,m_1 \frac{l_1}{I_1} (3\,k_1^2 - 1) + 6\,E\,\beta.$$

But $$k_1 = 1, \text{ and } m_1 \frac{l_1}{I_1} (3\,k_1^2 - 1) = 2\,m_1 \frac{l_1}{I_1}.$$

Moving this term to the left side we get:

$$M_0 \frac{l_0}{I_0} + 2\,M_1 \frac{l_0}{I_0} + 2\,(M_1 + m_1) \frac{l_1}{I_1} + M_3 \frac{l_1}{I_1} = 6\,E\,\beta.$$

Mr. Slettum.

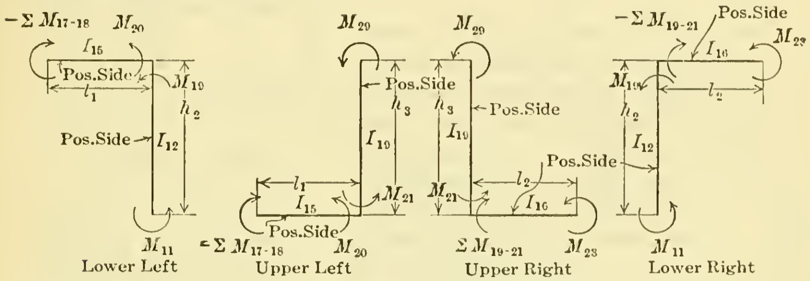
But $M_1 + m_1 = M_2$, therefore we have, in general, for such cases:

$$M_0 \frac{l_0}{I_0} + 2\,M_1 \frac{l_0}{I_0} + 2\,M_2 \frac{l_1}{I_1} + M_3 \frac{l_1}{I_1} = C_0 + C_1 + 6\,E\,\beta \ldots\ldots (4)$$

This is the three-moment equation as applied to frameworks.

From Equation (2) we see that β is positive when the angle between the chords of the deformed members measured on the positive side of the beam is increased, and negative when the same angle is decreased.

Choosing the under side of $P_9\,P_{10}$ as the positive side, we have (see Fig. 37 and also Fig. 31):



Note that in these sketches the moments are shown applied to the members, and not to the panel points.

FIG. 37.

For "lower left":

$$\Sigma\ M_{17\text{-}18} \frac{l_1}{I_{15}} + 2\,M_{20} \frac{l_1}{I_{15}} - 2\,M_{19} \frac{h_2}{I_{12}} + M_{11} \frac{h_2}{I_{12}}$$

$$= C_{15\,(9)} + C_{12\,(6)} - 6\,E\,\beta_{10\,(2)} \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

For "upper left":

$$\Sigma\ M_{17\text{-}18} \frac{l_1}{I_{15}} + 2\ M_{20} \frac{l_1}{I_{15}} - 2\ M_{21} \frac{h_3}{I_{19}} + M_{29} \frac{h_3}{I_{19}}$$

$$= C_{15\,(9)} + C_{19\,(14)} - 6\,E\,\beta_{10\,(3)} \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

For "upper right":

$$M_{29} \frac{h_3}{I_{19}} - 2\,M_{21} \frac{h_3}{I_{19}} - 2\ \Sigma\ M_{19\text{-}21} \frac{l_2}{I_{16}} - M_{23} \frac{l_2}{I_{16}}$$

$$= C_{19\,(14)} + C_{16\,(11)} - 6\,E\,\beta_{10\,(3)} \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

For "lower right":

$$M_{11} \frac{h_2}{I_{12}} - 2\,M_{19} \frac{h_2}{I_{12}} - 2\,\Sigma\,M_{19\text{-}21} \frac{l_2}{I_{16}} - M_{23} \frac{l_2}{I_{16}}$$

$$= C_{12\,(6)} + C_{16\,(11)} - 6\,E\,\beta_{10\,(2)} \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

In these equations the first subscript for C refers to the member, and the second subscript, which is in parentheses, refers to the panel point for which C is to be figured. In the same way, the first subscript

Mr. Slettum. for β refers to the panel point, and the second subscript to the story of the frame. (See Fig. 31.)

From Fig. 38 it is seen that the positive side of the members in one panel is the inside, and in the next panel the outside, the inside of one panel being the outside of its neighbor. It would be necessary, therefore, to assume the positive side of one member and then go through the whole frame and mark the positive side, as shown in Fig. 38.

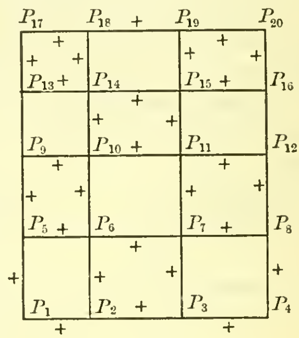


FIG. 38.

This would be very likely to cause some confusion, but the difficulty can be overcome simply by changing the signs throughout in the equations for "upper left" and "lower right". It is easily seen that this is equivalent to changing the positive side in "upper left" and "lower right", so that the inside of the panel to which the two members belong, in all cases, will be the positive side. The resulting equations for the point, P_{10}, therefore, are:

For "lower left":

$$\Sigma M_{17-18}\frac{l_1}{I_{15}} + 2\,M_{20}\frac{l_1}{I_{15}} - 2\,M_{19}\frac{h_2}{I_{12}} + M_{11}\frac{h_2}{I_{12}}$$
$$= C_{15\,(9)} + C_{12\,(6)} - 6\,E\,\beta_{10\,(2)} \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

For "upper left":

$$-\Sigma M_{17-18}\frac{l_1}{I_{15}} - 2\,M_{20}\frac{l_1}{I_{15}} + 2\,M_{21}\frac{h_3}{I_{19}} - M_{29}\frac{h_3}{I_{19}}$$
$$= -C_{15\,(9)} - C_{19\,(14)} + 6\,E\,\beta_{10\,(3)} \ldots\ldots\ldots\ldots\ldots (10)$$

For "upper right":

$$M_{29}\frac{h_3}{I_{19}} - 2\,M_{21}\frac{h_3}{I_{10}} - 2\,\Sigma M_{19-21}\frac{l_2}{I_{16}} - M_{23}\frac{l_2}{I_{16}}$$
$$\doteq C_{19\,(14)} + C_{16\,(11)} - 6\,E\,\beta_{10\,(3)} \ldots\ldots\ldots\ldots\ldots (11)$$

For "lower right":

$$-M_{11}\frac{h_2}{I_{12}} + 2\,M_{19}\frac{h_2}{I_{12}} + 2\,\Sigma M_{19-21}\frac{l_2}{I_{16}} + M_{23}\frac{l_2}{I_{16}}$$
$$= -C_{12\,(6)} - C_{16\,(11)} + 6\,E\,\beta_{10\,(2)} \ldots\ldots\ldots\ldots\ldots (12)$$

As the rule of signs now is the same for all panels, the positive sign always being the inside of the panel, we can write the following general equation: Let P_n, P_{n+1}, and P_{n+2} be the consecutive supports from left to right of any two members of a frame forming a lower left, upper

Mr. Slettum.

left, upper right, or lower right corner. Let M_n, $M_{n+1 \text{ left}}$, $M_{n+1 \text{ right}}$, and M_{n+2} be the corresponding moments. Let l_m and l_{m+1} be the length of the corresponding members. Let I_m and I_{m+1} be the moments of inertia of the corresponding members.

Let $\beta_{n+1(r)}$ be the change in angle between the two members.

Let C_n and C_{n+1} be the influence of external load on the corresponding members.

Then we have:

$$M_n \frac{l_m}{I_m} + 2\,M_{n+1 \text{ left}} \frac{l_m}{I_m} + 2\,M_{n+1 \text{ right}} \frac{l_{m+1}}{I_{m+1}} + M_{n+2} \frac{l_{m+1}}{I_{m+1}}$$
$$= C_{m(n)} + C_{m+1(n+2)} + 6\,E\,\beta_{n+1(r)} \ldots\ldots\ldots\ldots (13)$$

This equation is general, and the rule of signs is as follows:

The continuity moments, M_n, M_{n+1}, etc., enter with positive sign when producing tension on the positive side of the member, that is, the inside of the panel to which the member belongs. The quantities, C_m and C_{m+1}, enter with positive sign when the external loads act in such a way as to produce tension on the negative side of the member, that is, the outside of the panel to which the member belongs.

The quantity, $\beta_{n+1(r)}$, is positive when the angle between the two members, measured on the inside of the panel to which the two members belong, becomes greater; or, in short, follows the same rule as for the ordinary continuous beam.

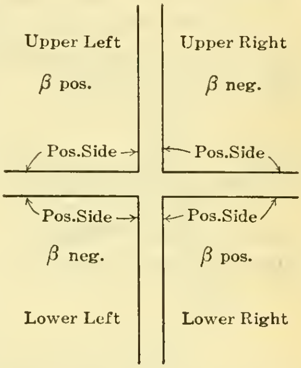


FIG. 39.

To make the case absolutely clear, the positive sides of members and the sign for β have been shown in Fig. 39. The sign for β assumes that the frame is supposed to deflect in the positive direction, that is, from left toward right.

Number of Unknowns and Equations Required to Solve the Problem.—At each panel point there are as many unknown bending moments as there are members meeting at the point, less one, this latter being equal to the algebraic sum of the others and opposite in direction.

Besides, we have an unknown deflection, $\delta = \beta\,h$, for each story of the frame.

This gives a total number of unknowns equal to twice the number of members in the frame, less the number of panel points, plus the number of stories.

As already stated, we can write a three-moment equation for any combination of two members meeting at a point. However, only three

Mr. Slettum.

—or in general the number of members meeting at a point, less one—are independent of each other.

To illustrate this, let us again consider the point, P_{10}. Equation (9) expresses the relation, $\delta = j$ (see Fig. 40); Equation (10) expresses the relation, $j = \epsilon$; and a combination of the two will express the relation, $\delta = j = \epsilon$.

Equation (11) expresses the relation $\epsilon = \lambda$. A combination of Equation (11) with Equations (9) and (10), therefore, will express the relation, $\delta = j = \epsilon = \lambda$. Thus it is seen that the elastic relation between the members meeting at a point is fully expressed by three equations, and that, by writing the three-moment equation for any other combination of members, we write an equation which is nothing but a combination of the three equations, and accordingly dependent on these.

Therefore, for each panel point, we have a number of equations, of the general form given by Equation (13), equal to the number of members meeting at the point, less one. These will be termed "elastic equilibrium" equations.

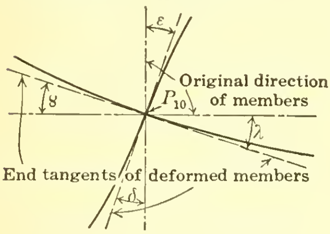


FIG. 40.

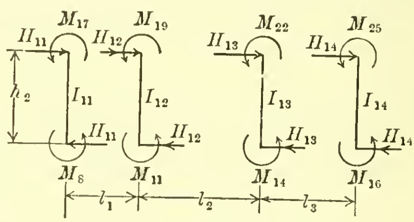


FIG. 41.

Let us now consider, for example, the second story. Let H_{11}, H_{12}, H_{13}, and H_{14}, be the shears in the respective columns. Assume the columns to be cut at the floor levels, and the internal forces applied, as shown in Fig. 41.

As the system of forces is in equilibrium, we have:

$$\begin{aligned} M_8 + M_{11} + M_{14} + M_{16} + M_{17} + M_{19} + M_{22} + M_{25} \\ = h_2 (H_{11} + H_{12} + H_{13} + H_{14}) \\ = h_2 \ (H_V + H_{IV} + H_{III}) = h_2 \ \delta_{II}, \end{aligned}$$

where δ_{II} is the external shear in the story.

Similar equations can be written for each story. These equations can be written in the following general form:

$$\Sigma \ M \text{ cols.} = h \ \delta \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

where $\Sigma \ M$ cols. = the algebraic sum of the continuity moments of the columns in any one story.

These will be termed "static equilibrium" equations. We have now a total number of equations as follows:

First, a number of "elastic equilibrium" equations equal to twice the number of members in the frame, less the number of panel points; and second, a number of "static equilibrium" equations equal to the number of stories in the frame. The total number of equations, therefore, equals twice the number of members in the frame, less the number of panel points, plus the number of stories, which is the number required to solve the problem. Mr. Slettum.

Solution of the Problem.—Considerable time can be saved by following always a certain method in solving problems of this nature. The following is proposed:

(*a*) Make sketches of the structure, and show the quantities, as in Figs. 29 and 30. In Fig. 30 it is not necessary to show the deformed members themselves, but only their chords.

(*b*) Tabulate the values of l, h, I, $\frac{l}{I}$, $\frac{h}{I}$, and C.

(*c*) Write the equations with coefficients for the unknowns and the constants given by letters. For each story write the equation containing the term $6\,E\,\beta$ by itself, and in the other equations containing that term, substitute its value, as given by this equation. Arrange the unknowns in numerical order, and place all constants on the right-hand side of the equation.

(*d*) Tabulate the numerical values of the coefficients for the unknowns and of the constants.

(*e*) Eliminate the unknowns.

(*f*) Find the unknowns by substitution.

The deflections are calculated from the general formula:

$$\delta = \beta\,h \qquad (15)$$

where δ = deflection of any single story.

The total deflection of the frame is:

$$\Delta = \Sigma\,\delta = \Sigma\,\beta\,h \qquad (16)$$

After the continuity moments have been calculated, each member may be assumed to be cut and freely supported at the panel points, and loaded with the external loads and the continuity moments. The further calculation is then as for an ordinary beam on two supports.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

PEARL HARBOR DRY DOCK

Discussion.*

By Frederic R. Harris, M. Am. Soc. C. E.

Frederic R. Harris,† M. Am. Soc. C. E. (by letter).—The writer is much interested in this paper on the Pearl Harbor dry dock, giving as it does a description of the geology of the location, the history of the design, the difficulties encountered in the construction, and the evolution of the plan on which the work is now being done. Mr. Harris.

In the latter part of March, 1913, shortly after the disastrous collapse of February 17th, the writer, under orders from the Secretary of the Navy, visited Pearl Harbor with Mr. Stanford, and spent 11 weeks investigating and examining the work, and inquiring into the causes of failure. Mr. Stanford was then, as he is now, Chief of the Bureau of Yards and Docks, and had many duties and responsibilities other than this particular work, so that, during the 2 weeks that he remained on the Island, much of the planning for a systematic investigation, together with the charge of the actual work thereof, was directly in the writer's hands. After Mr. Stanford's departure, in addition to this work, the writer, with E. R. Gayler, M. Am. Soc. C. E., the officer in charge, and Mr. Gordon, was a member of a Board to investigate the foundation conditions of the site, and this work was carried on independently of the principal investigation, although many of the data obtained for the latter were used in connection with the work of the Board.

Geological.—In addition to the comment in the paper on the geological formation, the writer would like to bring out the impression, made by conditions on the Island of Oahu and other islands of the Hawaiian group, on an engineer familiar with work on the main con-

* Discussion of the paper by H. R. Stanford, M. Am. Soc. C. E., continued from September, 1915, *Proceedings*.

† Philadelphia, Pa.

Mr. Harris.

tinent, where his experience would furnish little or no precedent to guide him in an understanding of the Hawaiian soil and foundation conditions. As an illustration, ordinarily, in the older main-continent formation, when the over-burden of earth and soils has been removed and rock is encountered, and after proper precaution has been taken to be sure that it is not a boulder, the assumption is well warranted that it is ledge rock and extends indefinitely downward. In the Hawaiian Islands, frequently a volcanic rock is encountered, and soils or disintegrated rock are found below it. Geologically, the islands are very recent, and earth or soil similar in character to the soils of the continent would yield and compact considerably under a foundation pressure, which, in the United States, would be considered thoroughly safe, a probable and logical explanation being that such surface soils, or soils comparatively near the surface, have never been subjected to the great pressures of an over-burden, later moved by erosive and weathering action. They are so recent geologically that there has been no opportunity for the working over with mechanical and chemical changes that will occur with the passing of future ages.

Borings.—On page 1127,* and in what follows, Mr. Stanford has mentioned the exploration borings, made under the direction of C. W. Parks, M. Am. Soc. C. E., in 1908, and the check borings, which were made under the writer's directions. In further explanation of this information, the writer, for a long time, in common with many other engineers, has believed that it is frequently difficult fully to appreciate and understand subsoil conditions and formations from the information obtainable and available by the wash-boring method of exploration, especially when the investigation attempts to bring out the compactness and imperviousness of the soils in place, the difficulty being emphasized by the recent volcanic formation of these islands and the extreme uncertainty of properly classifying materials brought up by wash-borings where experience and judgment would be predicated on continental formations.

Previous to sailing for the Islands, the writer had a conference with A. L. Parsons, M. Am. Soc. C. E., on this matter, and, on arriving, received from him some advertisements and descriptive matter relative to the Calyx double-core barrel drill for soft ground exploration. An attempt was made to secure such a drill, but the nearest one was then in El Paso, Tex., and to obtain it would have involved considerable time and delay. A smaller rotary drill of this type was in use by the army engineer in Honolulu, but it was not furnished with a double-core barrel, so that it was attempted to secure solid cores by the driven-pipe method, as described by Mr. Stanford on page 1128.* The writer was satisfied that the results being obtained closely represented the subsoil conditions, but the earlier reports from Mr. Gayler, and

* *Proceedings*, Am. Soc. C. E., for May, 1915.

Mr. Harris.

the opinion of the contractor's engineer, Francis M. Smith, M. Am. Soc. C. E., appeared to be that the soils underlying the work were of a semi-liquid or plastic character, and that the failure of the work had been caused by liquid mud pressure, that is, the pressure of the material composing the banks on the lower so-called lava mud. Figs. 8 and 9 are typical of the cores obtained from the driven pipe. A great many cores and samples obtained in the boring operations show no evidence of plasticity or a liquid condition, but Mr. Smith believed that possibly the driving of the core barrel ahead of the casing might have compacted the material, especially the so-called lava mud, and driven out the water, so that although the obtaining of cores was proceeded with by the driven-pipe method, arrangements were made to use the rotary Calyx drill. Four borings on the bank were made by this drill, the object being to check up by this method of boring, which did not involve compacting the cores. The cores were inspected as they were secured, and their specific gravity or weight submerged in sea water was taken, the object of which will be made clear hereinafter.

In addition to obtaining the solid cores and their weight in sea water, it was decided to investigate the porosity or perviousness of the subsoil, especially to locate, if possible, continuous impervious strata by permeability tests. In some recent core borings in ledge rock, the perviousness or leakage has been ascertained with two plungers or pistons, closely fitting the bore, by supplying water under pressure to the space between them. As the operations were carried on at various elevations, it was, of course, impossible to apply this method, and the following expedient was adopted: Directly after withdrawing the driven-pipe core barrel, the casing pipe was filled with water to a height of several feet above tide level, thus subjecting the subsoil at the foot of the casing pipe to a pressure or head of several feet of water, and then the loss of water in the casing pipe was observed for a time. As the joints in the casing pipe were not leaded, undoubtedly, in many instances, slight leakage occurred, so that very small outflows of $\frac{3}{8}$ in. per min. or less were considered as representing practically an impervious soil. These observations were continued and recorded. The result was a strong indication that the so-called lava mud and this same material mixed with some coral was impervious and continuous over the entire area, forming a complete water-tight diaphragm. The coral materials above this lava formation were somewhat pervious, and those below it were more pervious, and, in instances, quite freely water-bearing. The indications in all these observations were that the water present in the soils was under a ground-water or tide-water head equivalent to about that of tide level. It is stated by Mr. Stanford that "the character of the mud stratum has been of vital importance throughout

Mr. Harris. this work." Although, geologically, it might be termed "mud", its physical characteristics would be defined more clearly to an engineer by the designation "clay". Under the microscope, it was more granular, the contained water was hygroscopic, and it was deficient in aluminum, as compared with continental clays. In making and burning briquettes, in some instances they were partly vitrified, and in others powdered and disintegrated. Samples immersed in water did not in all instances show the characteristics of continental clay, nevertheless, they did not dissolve readily and break up, but preserved their shape to such an extent that it was possible to immerse them in water and obtain their specific gravity and still retain them as effective samples. Plate XLIV, made up from the 1913 borings, indicates the probable strati-

DRY DOCK, NAVY YARD, MARE ISLAND

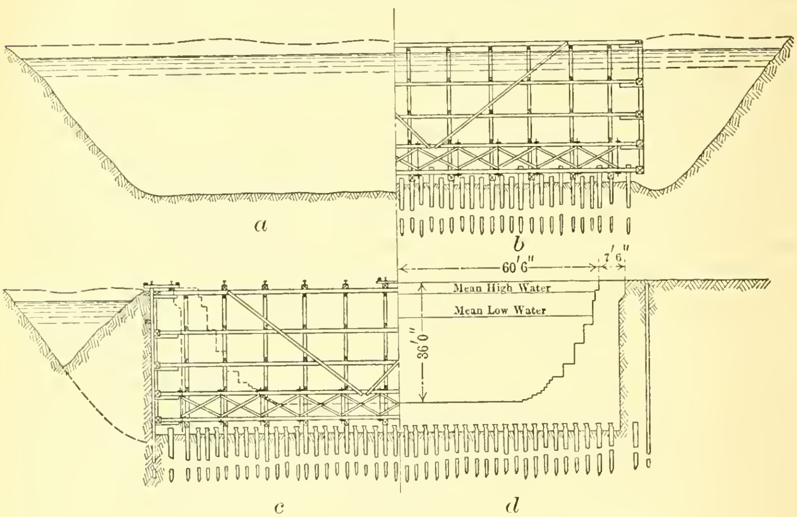
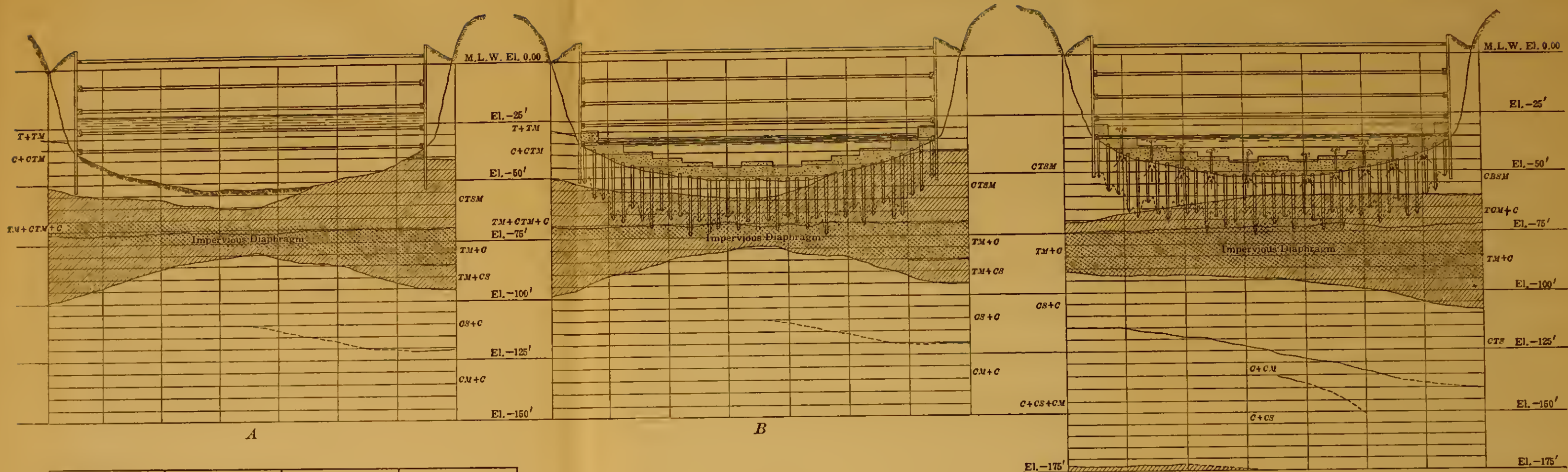
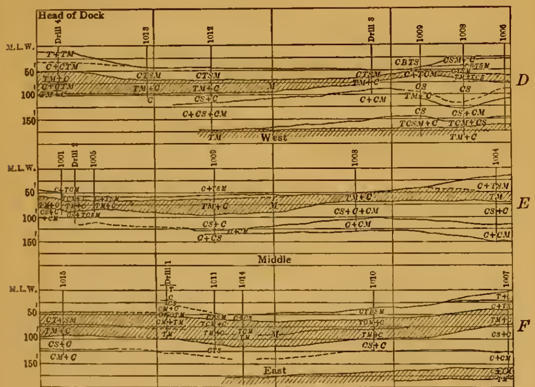


Fig. 21.

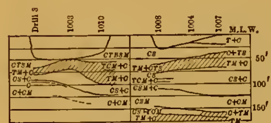
fication on the site of the structure. It is possible, of course, that the so-called lava mud stratum was not continuous and complete, but the borings appear to indicate that it was. The cross-hatched, shaded portion of the stratification is the apparently impervious stratum or diaphragm. It will be noted that at a considerably lower elevation, beyond the depth of direct interest in the investigation of this work, a somewhat similar stratum of great depth is found.

Design and Construction.—Mr. Smith, in charge of the work for the San Francisco Bridge Company, had successfully completed the dry dock at Mare Island, Cal., previous to his taking charge of this work, the method of construction being shown in Fig. 21, in four stages, which are largely self-explanatory. The work was completed by





LONGITUDINAL SECTIONS



TRANSVERSE SECTIONS

H *G*

STRATIFICATION INDICATED TAKEN FROM REPORT OF PROFESSOR T. A. JAGGAR JR., GEOLOGIST

C=Coral
T=Tuff
B=Basalt
S=Sand and Gravel
M=Lava Mud or Clay

PEARL HARBOR DRY DOCK

DIAGRAM SHOWING SUBSOIL CONDITIONS AND TYPICAL SECTION IN CRIBS I AND II FOR 1st, 2d AND 3d FAILURES

making the excavation, building large timber cribs, sinking these, driving around the periphery heavy timber sheet-piling, backing this up with the earth back-fill, and then pumping out the interior. A similar method was adopted for this dock, it being planned to use five sections or cribs, as explained by Mr. Stanford. Mr. Harris.

Section *A* of Plate XLIV is typical of subsoil conditions in Section I, and shows the operations there in the first attempt to unwater it, called Failure No. 1. The rising of the crib can be explained by the removal of the weight of water from its interior—the hydrostatic pressure on the under side of the diaphragm being resisted by the weight of the water left in the interior of the crib, the weight of the saturated soil above the diaphragm, and the excess weight of the crib itself over its buoyancy. After this attempt the condition was reviewed by a Board, and the writer first became familiar with conditions at Pearl Harbor by discussing them with a member of this Board. As described by Mr. Stanford, foundation piles of a certain length were specified, and the contractor was permitted to place concrete under water by the tremie method; then piles were driven and the concrete was placed in Section I, and an attempt was made to unwater this, all of which is described. A typical condition is shown by Section *B* of Plate XLIV, the hydrostatic pressure on the diaphragm being resisted by the same forces as those described for Section *A*, with the addition of the weight of the concrete submerged in sea water, and also by the anchorage effect afforded by such foundation piles as penetrated through the diaphragm and engaged by pile skin friction the soil materials below the diaphragm. The concrete itself forms a seal, complicating the action, and, in itself, irrespective of the existence of the impervious diaphragm some distance below subgrade, would bring about a new condition of affairs, because, without relief by bleeders or otherwise, the hydrostatic pressure on the under side of the concrete, in order to avoid uplift, would have to be counterbalanced by the weight of the water in the interior of the crib, the excess weight of the crib, the submerged weight of the concrete seal, and the anchorage effect of the foundation piling, in turn dependent on the submerged weight of the mass or prism of earth engaged by these foundation piles. As given by Mr. Stanford on page 1111*, the average length of the 1 966 piles driven in Section I was 23.78 ft., and although the resistance to up-pull for a single pile is dependent on the skin friction of the pile, or the actual mechanical anchorage in the case of piles with enlarged bases, in either case it is limited by the weight submerged in water of the conical but irregular masses of earth surrounding the pile that would have to be lifted; the anchorage effect in a group of piles driven closely together would be less than the sum of the individual effects of a like number of single

* *Proceedings*, Am. Soc. C. E., for May, 1915.

Mr. Harris. piles, and in turn would be limited by the irregular prism or mass of earth pinned together or engaged by pile skin friction, so that the exact effect of the anchorage piles would depend not so much on the average length of the piles as on the relation of the long piles to the average.

In making a check computation on Failure No. 2, the rising of the crib was noticeable with water removed from the interior to from 32.8 to 34.3 ft., and this would give the mean depth or thickness of the mass of earth lifted as 22 ft., as compared with an average length of piling of 23.78 ft.

The next failure, or rather the final collapse, occurred on February 17th, 1913, in Section II, the typical conditions of which are shown by Section *C,* Plate XLIV. As stated by Mr. Stanford, the average length of piles in this section was 20.53 ft., giving by similar computation the mean depth of the mass of earth lifted as 18.5 ft.

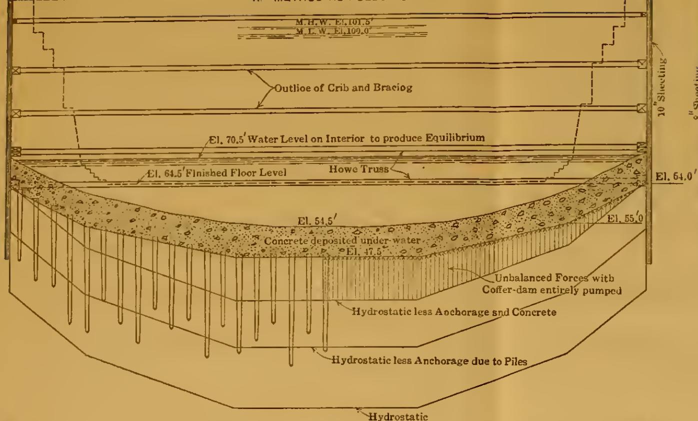
Referring to the sixty 2½-in. vertical drain-pipe bleeders described by Mr. Stanford, on examination of Section *C,* Plate XLIV, it will be readily seen that these merely served to drain the confined soil above the so-called lava mud diaphragm, and did not serve in any way to relieve the pressure under the diaphragm itself, and to that extent their effect toward averting failure was negligible, if of any use at all. The existence of the bleeders would serve to relieve more or less, and decrease, the hydrostatic pressure on the under side of the concrete seal, but with the existence of an impervious stratum or diaphragm some distance below sub-grade, by draining out the ground-water in the coral above the so-called lava mud, they might decrease the total weight of ground-water above the diaphragm, and to that extent decrease the resistance to uplift on account of the pressure below the diaphragm, and in this way possibly bring about an actual upward movement earlier than would be the case if these bleeders had not been put in. It is extremely doubtful whether any such conditions existed, but it is readily seen that the value of the bleeders was negligible, as they affected the hydrostatic pressure exerted on the under side of the so-called lava mud stratum, which was practically impervious.

In the computations referred to, 54 lb. was taken as the weight of the soil immersed in sea water; an average of actual observations gave 60 lb., but this was reduced by 10% for possible compression in securing samples by the driven-pipe method.

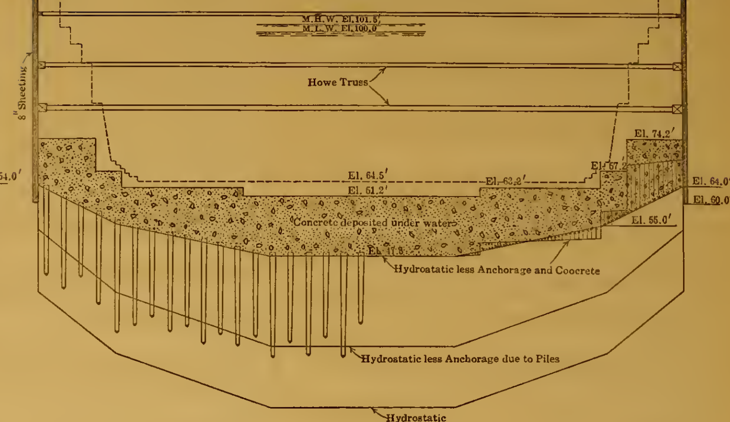
Suggested Plans.—In June, 1911, after the failure to unwater Crib I, and before the change was made specifying foundation piles, with permission to place a certain quantity of concrete under water by the tremie method, the writer became familiar with Pearl Harbor dry dock conditions, and on June 11th, in correspondence with P. L. Reed, M. Am. Soc. C. E., a member of the Board, suggested

PLATE XLV.
PAPERS, AM. SOC. C. E.
OCTOBER, 1915.
HARRIS ON
PEARL HARBOR DRY DOCK.

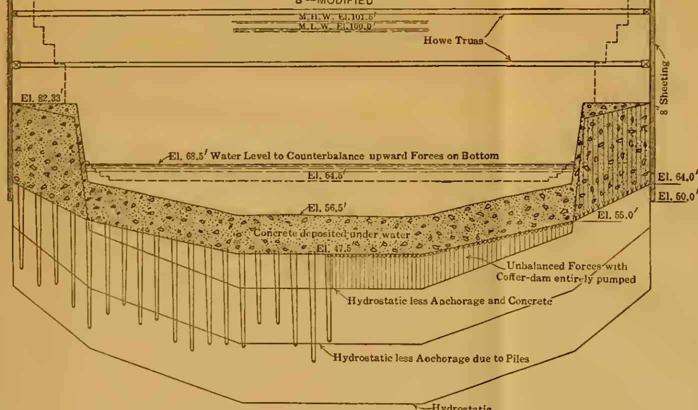
A.—METHOD AS FOLLOWED



C.—MODIFIED TO PERMIT COMPLETE PUMPING OF DOCK

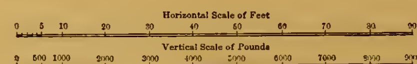


B.—MODIFIED



PEARL HARBOR DRY DOCK

METHOD OF CONSTRUCTION FOLLOWED BY CONTRACTOR
AND MODIFICATIONS PROPOSED TO CARRY IT OUT SUCCESSFULLY



that, if a tie were considered necessary for anchorage purposes, especially to provide for the construction condition, rectangular caissons be used instead of piles, in a manner somewhat similar to what was then being done on Dry Dock No. 4 at the New York Navy Yard, of the construction of which the writer was in charge, and called attention to the necessity, during this construction stage, of engaging by anchorage and a stratum or prism of earth of a mean depth of 35 ft., in addition to the submerged weight of an 8-ft. layer of concrete. Mr. Harris.

As stated by Mr. Stanford, the view was taken that the anchorage condition during the construction stages was a problem for the contractor, and it was doubtful whether any anchorage of foundation piles would be required for the completed dock. The Government, however. did provide for paying for foundation piles with an average penetration of 35 ft., with an arrangement for a price per linear foot, so that payment would only be made for lengths actually driven. The writer can state without hesitation that it was impossible to drive piles to a much greater average depth than was secured, which, in the case of Section I, was slightly in excess of 20 ft., and in Section II slightly less than 24 ft., so that perhaps the first suggestion for a change in plan was made by the writer when he proposed the substitution of pneumatic caisson piers and anchors. However, with the information available at that time, there was then apparently no serious reason for considering or adopting a costly change in plan.

In April, 1913, shortly after arriving in the Hawaiian Islands, and after becoming familiar with the conditions, the writer was strongly of the opinion that two general courses were available for the completion of this dry dock: First, following the lines up to that time proceeded with, by the construction in an open coffer-dam, holding down the structure during construction against uplift and flotation by added anchorage or weight, and, as such added anchorage could not be secured by driving deeper anchor or foundation piles, by either placing considerably more concrete by the tremie method under water before unwatering, or by adding weight in the form of rock ballast. The quantity of rock ballast that would have been required in one section was considerable, in fact, this plan seemed impracticable. The second plan was to adopt the familiar method often used in bridge pier construction, where the unwatering of the site and the building of an open coffer-dam is impracticable or undesirable; that is, the preparation of a level foundation and the sinking of the structure on this prepared foundation through the water without unwatering, that is, the use of floating coffer-dams or caissons. In February, 1910, while the writer was in charge of the dry dock construction at New York, he had suggested to Mr. Frederick Holbrook, of the Holbrook, Cabot and Rollins Corporation, the placing of the entrance of that dry dock by using a similar caisson, and had drawn up a tentative

Mr. Harris. plan for the work on these lines for the purpose of avoiding the construction of the usual open coffer-dam at the entrance. The plan was discarded as more expensive than that finally used. Further, in September, 1912, as an associate with William Barclay Parsons, M. Am. Soc. C. E., on a project for dry dock construction in South Brooklyn, the writer made studies, outline design, and estimates for a dry dock, using coffer-dams or caissons, building the dock in sections, and constructing these sections in one of the timber graving docks available at the Erie Basin plant. The writer had with him during his visit to Pearl Harbor one of these sectional sketches, and discussed the matter with Mr. Stanford, who then believed that it was not advisable or necessary to consider so elaborate a plan. He also discussed this plan with S. H. Hindes, M. Am. Soc. C. E., President of the contracting firm, and Mr. Smith. It is noted that Mr. Stanford, on page 1139,* states that the writer submitted two plans in a letter dated September 26th, 1913. It is believed that this is unintentionally misleading, as the writer, on his return from the Hawaiian Islands, at the end of June, 1913, in a report to the Secretary of the Navy, dated July 1st, 1913, and written some time before this in the Hawaiian Islands, states:

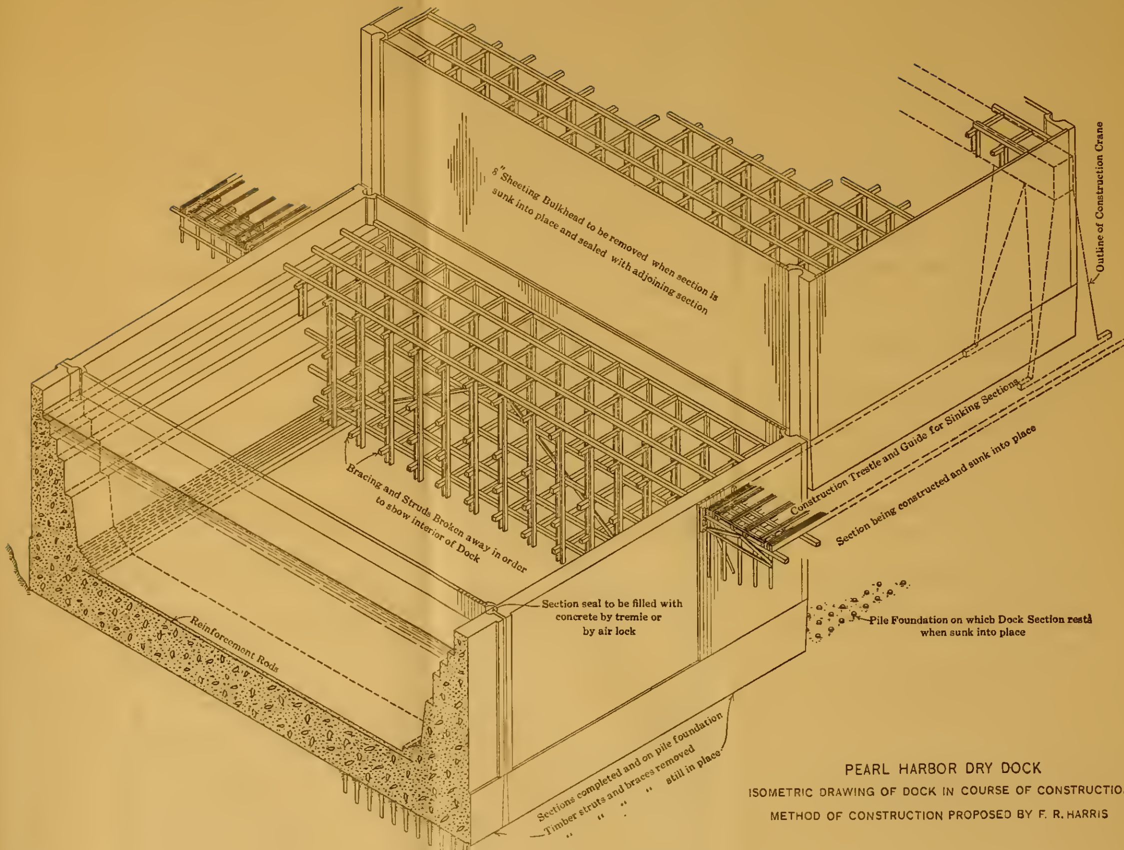
"In view of what has been developed on this work, it is quite probable that some method of construction dependent upon the use of either timber or reinforced concrete floating caissons would lend itself more readily to the construction of the dry dock on this site. I have in mind the possible dividing of the dock into ten sections, each caisson to be the full width of the dock and 100 feet long; concrete masonry work of the dock proper to be built in the dry in the interior of these boxes, the added weight assisted by weight of flooding water to sink these boxes or caissons to place upon the prepared pile foundation. In Sections 1, 3, 4, and 5, working chambers could be provided to be operated under air pressure, these chambers later filled with concrete surrounding the heads of piles already driven; in Section 2 all of the concrete masonry to be placed in the open, no working chamber or air pressure to be used, and the entire mass sunk in place as a gravity structure resting upon the prepared pile foundation, or the same method outlined for Section 2 to be used for Sections 1, 3, 4, and 5. This method of construction would permit of reinforcing with steel rods the floor or bottom of the dock so as to insure practically equal distribution of load over the dock bottom."

In the summary or letter of transmission, of July 1st, 1913, this statement was contained:

"With foresight and care it should have been possible, and still is possible, to employ the construction methods successfully that were employed previous to the failure, although certain modifications of the details of the design may be required to insure success of this method; such, for instance, as the placing of more concrete in the

* *Proceedings*, Am. Soc. C. E., for May, 1915.

PLATE XLVI.
PAPERS, AM. SOC. C. E.
OCTOBER, 1915.
HARRIS ON
PEARL HARBOR DRY DOCK.



PEARL HARBOR DRY DOCK

ISOMETRIC DRAWING OF DOCK IN COURSE OF CONSTRUCTION

METHOD OF CONSTRUCTION PROPOSED BY F. R. HARRIS

bottom, and the application of movable weight to the bottom, all sufficient in amount to restore equilibrium during construction." Mr. Harris.

Immediately on the writer's return from the Hawaiian Islands, he started the development of the plan of the sectional floating caisson, recommendation of which was made in his report of July 1st, and on its completion it was forwarded on September 26th, 1913, the date referred to by Mr. Stanford. Only one plan was submitted—that of building the dry dock by sectional floating caissons—this plan, as may be seen by reference to Plates XLVI and XLVII, being practically a sketch or pictured drawing. The details were not worked out, nor were any complete computations made. The other matter referred to as a second plan by Mr. Stanford was merely a graphical demonstration of the method used by the contractor, the probable cause of failure by unbalanced forces, and a suggestion as to how equilibrium could be brought about by the addition of under-water concrete, all shown by Plate XLV. The adoption of the floating caisson plan was urged in this report as more certain and expeditious, and as a plan that, in view of the conditions developed by the failure of February 17th, 1913, would remove considerable of the danger and hazard. The time from July 1st to September 26th was taken up in preparing the picture sketches and consulting with several engineers and contractors of experience, in order to obtain from them their views as to the expense of this type of construction. The writer did not then hold the view, nor does he now, that anything so very new or remarkable was being recommended, but stated in his report:

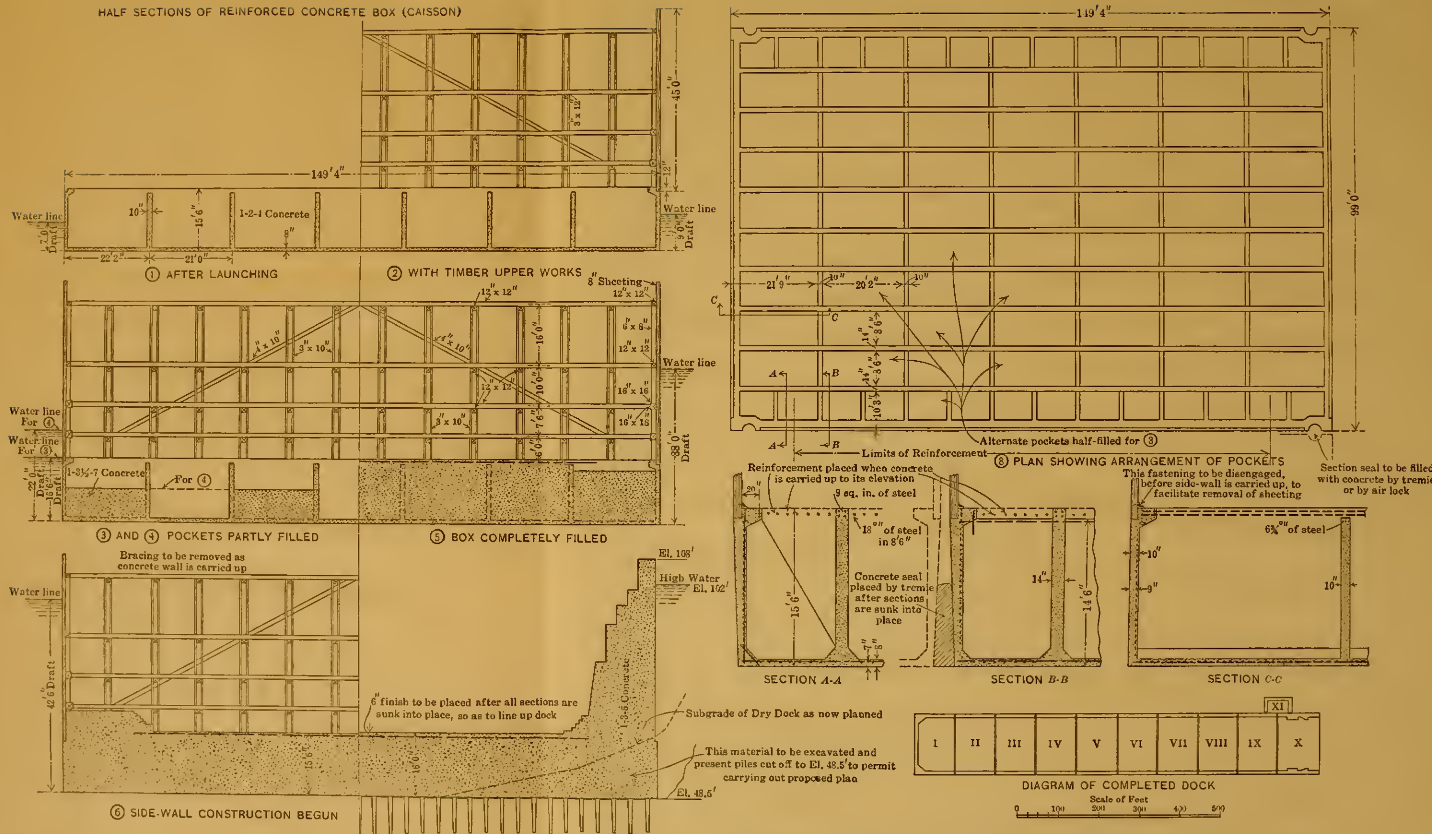
"It is believed that this is an untried method for the construction of dry docks, but a method that has been frequently used for the placing of bridge piers, although probably not on so large a scale nor involving the placing of so many separate sections to be later connected and sealed."

Up to October, 1913, the writer had not been informed of the employment of the late Alfred Noble, Past-President, Am. Soc. C. E., by the Government, or that he had visited the Hawaiian Islands. Shortly after his return, early in October, 1913, Mr. Noble communicated with the writer and informed him of his visit, and that he was making some studies in the matter. The writer, at his request, forwarded to him the studies contained in the supplementary report of September 26th, Mr. Noble informing him that he already had his report of July 1st and had taken that out to Pearl Harbor with him. Mr. Noble never claimed that his recommendation, that the sectional floating caisson plan of construction be used at Pearl Harbor, was other than an elaboration and development of the recommendation of the writer, for how could he, as he took the writer's report of July 1st, containing the suggestion of this method, to Pearl Harbor

Mr. Harris. with him? In order to correct any erroneous impression, the writer would state that the information given, that Leonard M. Cox, M. Am. Soc. C. E., had in 1911 proposed the adoption of this method of construction, is news to him, as previous to his visit to Pearl Harbor and to submitting either the report of July 1st or the supplemental report of September 26th he had not found any such suggested plan among the papers and data given to him in connection with the Pearl Harbor investigation, the complete records of which were supposed to have been placed at his disposal, nor did he receive any intimation that such a plan had been proposed or existed.

The construction of bridge piers by this method is not new; it was applied successfully in breakwater construction in Zeebrugge, Belgium, in 1905; in Barcelona, Spain, in 1906; in Touapse, on the Black Sea; in Talcahuana, Chili, in 1908; in Bilbao and Bizerta; and in the United States under patent secured by W. V. Judson, M. Am. Soc. C. E., in Algoma, Wis., etc.; and on quay wall construction in Rotterdam; Norresundby, Denmark; Passau, on the Danube; and on a large and extensive scale in connection with pneumatic pressure and diving-bell work at Antwerp, Belgium. A dry dock built at Toulon, France, was constructed in a large steel caisson with a working chamber beneath; an extension to the dry dock at Dunkirk, France, by a single pneumatic caisson; and the dry dock at New York Navy Yard, constructed under the writer's supervision and design, by surrounding the site with a cut-off wall sunk into place in pneumatic caissons with central piers or anchors placed by the same method. The writer believed that the method recommended by him in the report of July 1st, 1913, was new and untried for dry dock construction, and on this assumption filed an application with the United States Patent Office on October 16th, 1913, and a patent was granted to him on April 6th, 1915. In making preliminary application for a patent, in August, 1913, the writer went on record with the Patent Office to the effect that the Government was to be granted free use of the patent for dry dock construction.

After the submission of the report referred to, the writer had no further connection with Pearl Harbor dry dock matters until January, 1914, when he was consulted relative to taking charge of the completion of the construction of this dry dock, on the same plan and construction methods being followed out previous to the failure. In his opinion this gave little promise of a successful completion of the dock, and involved great hazard and uncertainty. He then for the first time had the opportunity of examining the report and recommendations made by Mr. Noble, and was much gratified and reassured to ascertain therefrom that such a careful and conservative engineer held the same views, and had recommended the adoption of the sectional floating caisson construction for the dry dock. After con-



DETAILS OF SUCCESSIVE STEPS IN CONSTRUCTION OF PEARL HARBOR DRY DOCK

METHOD OF CONSTRUCTION PROPOSED BY F. R. HARRIS

siderable negotiation and conference, Mr. Cox and the writer were designated to prepare the necessary new studies and plans for the construction of the work. The writer devoted himself to the development of a suitable sectional floating caisson design, Mr. Cox confining his attention to some means of adhering to the original plan by using, for holding-down purposes at the sides, heavy concrete blocks, and at the center a steel ballast tank, very similar in general proportions and dimensions to the ballast tanks of the interior of the coffer-dam boat shown in Figs. 13 and 14. Mr. Harris.

Mr. Cox did not at that time inform the writer of having suggested or recommended the adoption of the sectional floating caisson method of construction, and in fact at that time was opposed to the adoption of this plan, although the writer has been more recently informed that at some time he discussed with the contractor's engineer, Mr. Smith, and made a rough sketch of a reinforced concrete floating dry dock that might be sunk as a unit and filled with concrete.

Mr. Stanford states on page 1139* that the plan recommended by the writer was to construct the base or hull for each unit on launching ways. The writer believed that this would be the most economical and expeditious method of construction, but did not confine his recommendations within such narrow limits. In fact, in his applicaton for the patent, which was granted, he states:

"The floating box or caisson (*a*) shown in Fig. 1 may be constructed in the dry on inclined ways and then launched into the water, or it may be built in a shallow, excavated basin, this being then flooded, or by any other method whereby the floating caisson can be built in the dry and subsequently floated in the water to its proper position. Each box is then to be surmounted by a timber, concrete, or steel extension or lagging * * *."

The writer trusts that he may be pardoned for reference in this discussion to the patent granted to him. It was not taken out with any mercenary motive, nor with a view of any monetary return. He has honestly been under the impression, in so far as the adopted design used in the construction of this dry dock is concerned, that the recommendation for the use of the sectional floating caisson method of construction came from him. It would appear that it certainly first came from him after the failure of February 17th, 1913, when Mr. Stanford and the writer were instructed to investigate the failure of this work and recommend a means of overcoming such difficulties as might be found to exist, and when there was occasion to recommend a radical departure in the design. The writer believes that considerable remains to be done in carrying out the projected construction, from which by no means all the hazard and uncertainty have been removed. The construction of these large floating sections, the placing

* *Proceedings*, Am. Soc. C. E., for May, 1915.

Mr. Harris. and removing of the floating coffer-dam, and the successful lowering of these sections to place on a properly prepared foundation involve the greatest difficulty, and call for considerable ingenuity, especially in order to avoid great and severe stresses in landing a section and bringing it to an equal bearing over the entire foundation area. Even the greatest care and foresight may perhaps not forestall an accident, resulting in the loss of an entire section and the necessity for its subsequent removal. In his original recommendation, the writer suggested that the foundation be prepared by sawing off piles with an under-water saw arbor, and that each section be provided with a base of 2 or 3-in. soft pine planking, so that when settled to place on the piles, a more or less even bearing would be secured by local crushing. He understands that the plan now adopted contemplates cutting off the foundation piles about 1 ft. below sub-grade and covering the entire area with broken stone, to be leveled off under water.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE ACTION OF WATER UNDER DAMS

Discussion.*

By Messrs. H. B. Muckleston, Warren B. Travell, Malcolm Elliott, James B. Hays, W. P. Creager, and Edward Wegmann.

Mr. Muckleston.

H. B. Muckleston,† M. Am. Soc. C. E. (by letter).—This very interesting paper is most valuable, especially the curves presented as the result of actual experiment, as compared with those derived from purely theoretical investigation. The problem is susceptible of rigid mathematical analysis, if certain assumptions are made. These are: First, that the permeable base extends to an infinite distance in each direction in a horizontal plane; second, that the depth of the stratum is infinite; third, that it is uniform in permeability throughout and in any direction; and fourth, that the flow is capillary. Under these conditions, if a cut-off wall is considered as existing under the center of the impermeable base, of any depth, a, and the base width is $2b$, then the contours of equal pressure are the series of confocal hyperbolas:

$$\frac{x^2}{\cos.\ \beta} - \frac{y^2}{\sin.\ \beta} = c^2$$

where

$$c^2 = \frac{x}{b^2 - a^2}$$

and the lines of flow normal to them are the series of confocal ellipses :

$$\frac{x^2}{\cos.h\ \alpha} + \frac{x}{\sin.h\ \alpha} = c^2.$$

It is not necessary for the cut-off wall to be at the center, but if it is

* This discussion (of the paper by J. B. T. Colman, Assoc. M. Am. Soc. C. E., published in August, 1915, *Proceedings*, and presented at the meeting of September 15th, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Calgary, Alberta, Canada.

Mr. Muckleston. placed anywhere else, the equivalent depth at the center must be calculated from the equation.

It will be interesting to consider why the experimental results differ so widely from the theoretical. In the first place, the areas exposed to pressure between the ends of the box and the toe and heel of the experimental dam are very small in proportion to the base width of the dam; second, the depth of the permeable stratum is small compared to the base width; and third, the tail-water surface was somewhat above the sand level, whereas the theoretical hypothesis requires it to be at that surface.

The first and third conditions are probably responsible for the observed entry and exit heads, and the second is the cause of the peculiar shape of the pressure curves. The latter is the more noticeable when the experiments with the cut-off wall are considered, especially in those with the 3-ft. wall, where the throttling effect of the narrow passage is clearly seen. This effect is evident in all the plotted experiments, and it is very probable that had the observed pressure points been more closely spaced, island pressure contours would have been detected, for example, in Fig. 2, Exp. No. 38, Obs. No. 4, April 19th, 1914, between Points 19 and 29.

Another effect of the first condition is noticed most strongly in Fig. 7, Exp. No. 20, Obs. No. 3, March 24th, 1914, where the pressure contours up stream from the cut-off are seen to be nearly parallel and approximately horizontal. The sharp downward curvature of the 2.50 contour is also noticeable, and is probably due to its very close proximity to the foot of the wall and the constricted opening under it.

Although the assumed conditions of the theoretical calculations are probably very far from those obtaining in actual practice, it is believed that the conditions of the experimental determinations are about equally far in the opposite direction, the truth lying somewhere between them.

Mr. Travell. WARREN B. TRAVELL,* ASSOC. M. AM. SOC. C. E. (by letter).—This interesting paper bears directly on a question which is, not only very important, but concerning which prominent engineers of the present day have radically different opinions—the question of uplift on the base of concrete or masonry dams of the gravity type.

There is no doubt that, in the past, many dams have been built without any allowance having been made in their calculations for uplift, which is equivalent to taking the upward pressure on the base of the dam as zero, although a few engineers are insisting that the uplift should be taken at 100%, or full hydrostatic pressure.

These experiments constitute a valuable addition to our scanty knowledge of the subject, but it must be admitted that they were made

* Northampton, Pa.

on a rather small scale, and it is to be hoped that they will be added to by others, so that before long sufficient data will be furnished whereby engineers may come to an agreement in this matter. Mr. Travell.

It is noted that, in his experiments, the author used three different lengths of bases, namely, 4 ft. 3 in., 6 ft. 3 in., and 8 ft. 3 in., and on these short lengths as a foundation, he proceeds to draw conclusions applicable to all lengths of bases up to 100 ft.

The values of the constant b in the formula deduced for pressure, are determined to be 1.49, 6.13, and 24.78, for these three base lengths, and from these data, values of b up to 2 587 are derived. This appears to be equivalent to attempting to draw a curve having given only three points in that curve. The length of the curve thus given is the difference between 24.78 and 1.49, or 23.29 units, and this curve is then extended beyond the farthest given point to a length of 2 562 units. In other words, the curve is extended to a distance 110 times the determined length.

In the first place, three points are not sufficient to determine a curve of this kind, even for the distance between these points, and the extension of the curve thus found, especially when carried out to such a length, is interesting as the record of a guess, but is hardly entitled to a place among scientific data. Some of the greatest engineering disasters have resulted from the drawing of extended conclusions based on limited tests, and too great emphasis cannot be laid on the warning against such practice.

When frequent repetitions of an experiment are made under the same conditions, the uniformity of the results determines largely the degree of confidence to which such experiment is entitled. Examination of the tabulated pressure heads, given in Table 2 (Plate XXVII), apparently shows considerable variation for experiments conducted under similar conditions. For instance, in Experiment No. 29, Observation No. 1 was made with water at a height of 2.46 ft. above the base, and, in Observation No. 2, this was slightly less, or 2.44 ft. Yet the pressure heads given in the latter observation are as much as 36% greater than those given in Observation No. 1.

Regarding the curves for upward pressures, without sheet-piling, as shown in Figs. 9, 10, and 11, it is not clear to the writer why there should be about 80% of the 5-ft. head at the heel of the 4.25-ft. and 8.25-ft. dams and only 58% at the heel of the 6.25-ft. dam. It would seem that this figure should be practically constant for any given head, irrespective of the length of base.

Also, in Figs. 12, 13, and 14, showing pressures with different lengths of sheet-piling at the heel, it appears strange that, comparing the curves for the 5-ft. head, the pressure should increase from 49% of the head for 1-ft. sheet-piling to 66% for 2-ft. sheet-piling and to 76% for 3-ft. sheet-piling. As the sheet-piling was made longer, a

Mr. Travell. decrease in pressures would naturally be expected, rather than an increase as shown in these curves.

It would be of great value to engineers if experiments similar to these were made in connection with the actual building of some masonry or concrete dams of the gravity type. Naturally, there would be a wide variation in the results of such experiments, depending on the porosity of the underlying foundation, the presence of seams in a rock foundation, the density of the concrete, and many other causes, but data thus obtained and given to the Profession would eventually settle the debated question of uplift.

Mr. Elliott. MALCOLM ELLIOTT,* ASSOC. M. AM. SOC. C. E. (by letter).—For several reasons, the equations developed from this series of experiments do not seem to be suitable for general use in determining the upward pressure on the base of a dam built on sand foundation.

There is no known law of physics or hydraulics which would indicate that the pressures under a dam will be expressed by an equation in the form of $P = 62.5\ [bs + c\ (d^s - 1)]$. The author has measured the pressure on the model dam, platted the points, and drawn through them a curve which he finds is approximately the graph of the equation stated. So far as the writer knows, there is no reason that the pressures under a dam should be expressed by the curve of the law of error, and the fact that, within certain limits, the pressures are approximated by this curve should not be taken to indicate that the equation is a general expression for the pressure for all heads and floor lengths.

The utmost that can be said for the author's equation is that it gives fairly well the pressures which will occur in a model of the same dimensions and founded on the identical material used by him.

To demonstrate that the equation is not generally applicable, it might be well to cite some experiments carried on recently under the direction of J. P. Jervey, M. Am. Soc. C. E., Major, Corps of Engineers, U. S. Army, and described in an article† by Capt. W. A. Mitchell, Corps of Engineers, U. S. Army. The model used in these experiments was very similar to that used by the author. The floor length was 9 ft., made up of nine concrete slabs, 1 ft. wide, the joints of each being caulked. The heads were from 0 to 5 ft. Instead of having a flat bottom, the box containing the sand and the model dam was recessed so as to allow about the same space under the piling as under the slab. The piling was at the heel, and was 2 ft. long. In these experiments, three different materials were used, the following description of which is quoted from Capt. Mitchell's article:

"'Sand' means washed river sand screened through a $\frac{1}{4}$-inch screen.

* Louisville, Ky.

† Professional Memoirs, No. 31 (January–February, 1915), Corps of Engineers, U. S. Army, Washington Barracks, D. C.

" 'Washed gravel' means ordinary river gravel washed and screened through a 2-inch, and retained on a ¼-inch, screen. Mr. Elliott.

" 'Gravel aggregate' means ordinary river-bottom mixture of gravel, sand, and mud at the site of Dam No. 14 [Ohio River], without any washing or screening whatever.

" 'Alluvial soil' means ordinary sedimentary deposit on the banks of the river, as was obtained from the field near the apparatus."

The pressures transmitted through these materials are given in Table 8, which was compiled by the writer from Capt. Mitchell's data, the pressures having been converted from inches in the piezometer tubes to square feet of pressure curve, so as to afford a comparison with the pressures as computed from the author's equation. To obtain the actual pressure, in pounds, the quantities in Table 8 should be multiplied by 62.5.

TABLE 8.—Comparison of Pressures Transmitted Through Various Materials.

Head, in feet.	Slope $\frac{H}{L}$	Observed Pressures, U. S. Engr. Experiments.				Pressures computed from author's equations.
		Sand.	Gravel aggregate.	Washed gravel.	Alluvial soil.	
0.125	0.0139	....	....	0.84	....	0.51
0.208	0.0231	....	....	1.32	....	0.75
1.333	0.1481	3.92	....	....	....	5.43
1.5	0.1667	...	0.56	....	....	6.00
2.5	0.2778	6.90	....	....	....	9.73
3.0	0.3333	....	1.00	....	0	11.5
3.5	0.3889	7.88	....	....	....	13.3
4.0	0.4444	8.21	....	....	0	14.2
5.0	0.5556	....	1.77	....	0	16.6
5.33	0.5922	....	...	....	0	17.8

The most striking point brought out by this comparison is the difference in pressures for different materials. This evidence does not seem to substantiate the author's Conclusion 6, namely:

"The porosity, effective size, and uniformity coefficient have little influence on the upward pressure exerted against the floor of a dam."

Notwithstanding the difference in pressures shown by these two independent sets of experiments, due apparently to differences in porosity, etc., the writer, for practical purposes, would be inclined to accept the author's conclusion, because the influence of porosity, etc., if any, is not well understood, and cannot be determined without large-scale experiments at every dam site, and even then it would not be certain that the model used would duplicate the actual dam. For instance, the space under the model dam and the outlet through which the water flows are necessarily restricted in area, though the sand under a dam may be very deep and the outlet unlimited in extent.

Mr. Elliott. Who can say that these differences between "laboratory" and actual conditions do not materially affect the pressures?

It is probably because of the uncertainty as to the influence of porosity, etc., that, as a rule, the upward pressures are computed as if these factors had no influence. The fact, however, that these two sets of experiments give different pressures would indicate that an equation derived from experiments on a small-scale model and fashioned after no recognized physical law, cannot be considered generally accurate.

It is not necessary to call in outside evidence, however, to discredit the author's equations. They convict themselves. Extend the curve, in Fig. 16, for the 4.25-ft. floor, and it will be seen that for $\frac{H}{L} = 1.75$, the area of the pressure curve will be about 33 sq. ft., giving a pressure of about 2 060 lb. It will be granted without question that in no case can the upward pressure exceed the full head acting over the entire base. For $\frac{H}{L} = 1.75$, this would amount to about 1 950 lb. Thus, if the equation is true, we would have the surprising condition of an upward pressure greater than the head, and the water in the tubes would stand higher than the water surface above the dam. For values of $\frac{H}{L}$ greater than those cited, the computed pressures increase very rapidly, and greatly exceed the total head. With the 6.25-ft. and 8.25-ft. floors, equally absurd results are obtained by extending the curves to $\frac{H}{L} = 1.7$ and $\frac{H}{L} = 1.6$, respectively.

It is the writer's opinion that the safest and most convenient assumption for the upward pressure and the location of its centroid is that there is a uniform loss of head from full head to zero throughout the distance the water travels in percolating under the dam. The distance traveled in the case of a dam with sheet-piling at the heel is approximately twice the length of the piling plus the width of the dam, and this sum divided by the head will give the hydraulic gradient or slope of the "piezometric line".*

Although this method may not be strictly correct, on account of making no allowance for differences in porosity, uniformity coefficient, and effective size, it will, so far as known, give results which are on the safe side. At the same time, the effects of porosity, etc., if any, are so vaguely understood that it is practically impossible for the designer to determine what they are.

Usually this method will give pressures greater than those recorded in the experiments referred to herein, but, in view of the uncertainties previously mentioned, it is not believed that the margin of safety will

* The method is explained very clearly by Mr. W. G. Bligh, in *Engineering News*, December 29th, 1910, p. 708.

be unreasonably large, and the experiments are not conclusive enough to justify a departure from a method which, in the past, so far as known, has proved safe. Mr. Elliott.

The action of water under dams is not limited to the upward pressure that it will exert on the base. A dam may be safe against overturning and sliding and yet fail on account of the erosion of the material under it, due to the flow of water through the porous material. This is perhaps the most uncertain of all conditions that must be considered in the design of a dam on a granular foundation. An investigation of the action of water under dams, to be thorough, should certainly deal, to some extent, with erosion.

JAMES B. HAYS,* JUN. AM. SOC. C. E. (by letter).—During the past 15 months the writer has been studying along just such lines as those followed by Mr. Colman in his paper, and it gives him great pleasure to examine the results obtained by others, which, in a large measure, corroborate those obtained by himself. Mr. Hays.

A few months ago he began to write a paper on the subject for presentation to the Society, but press of other work and the desire to make further investigations have prevented its completion.

The object of the writer's experiments was to determine the dimensions of an earthen dam to be constructed on a river bed of deep gravel. They were conducted in a large wooden tank, with an actual model of the proposed dam, constructed to scale, built on the gravel. Sheet-piling of various depths was interposed at different locations. Glass tubes communicating with the interior of the gravel mass showed graphically the pressures at the different points. The dam was placed about at the middle of the tank, and water was introduced on the up-stream side. Readings were taken showing the pressures at the different tubes for each of several different heads of water on the up-stream side. Seepage was also computed for each of the different heads.

In view of the writer's experience, he agrees with the author on the following points: (*a*), That there is a large loss of head where the water enters the gravel; and (*b*) that a practically uniform and relatively small loss of head per foot occurs horizontally through the main body of the gravel where its cross-section is uniform.

Regarding the loss of head where the water leaves the sand, or gravel, the writer's experiments and those of the author did not give the same results. However, this may be explained in either one, or both, of two ways.

The author's model was constructed so that the water entered the sand in a space 4 ft. wide, traveled under the dam in a space 5 ft. wide, and then, in the case of the 8-ft. floor, issued through a space 2 ft. 11 in. wide. It will be noted that the point of entry and the

* Boise, Idaho.

Mr. Hays. point of egress are constricted sections, relative to the space under the dam, which would naturally cause the water to be retarded, showing thereby a greater pressure head. The writer avoided these constrictions, and believes that his model more nearly represents the case in actual practice.

Then, again, where the shorter floor was used, the entire pressure head was not consumed. In some of the experiments using the longer sheet-piling, the base of the writer's model was longer in proportion to the head of water supplied, and without the constrictions mentioned, all the initial head was found to be consumed in friction, etc., before it issued from the down-stream side. Given a definite head of water, acting on a body of sand or gravel with known characteristics, a definite velocity will be produced. This, in turn, means a certain loss of head per linear unit and, by extending the length of the body of sand, or gravel, a point is reached where all the initial head is consumed. This is shown by the intersection of the hydraulic grade line with the body of the sand.*

It is the writer's belief that the head lost by the water entering the sand might not have been so great had the width of the section been 5 ft. or more, instead of 4 ft. However, his own experiments show a large loss of head at the entrance of the water into the sand, or gravel. This, in all probability, represents the entry head and the velocity head.

The author's Conclusion 2 might be explained by the fact that with a shorter floor he has produced a steeper hydraulic gradient, hence a larger head is lost, creating the increased velocity necessary.

In regard to Conclusion 5, does the author mean that the pressure increases with the increase of depth, or the decrease of depth?

In general, aside from the points mentioned, the writer's experiments agree with those of the author, and though they do not reach the same conclusions on some points, this is explained, in all likelihood, by the difference in construction of the models.

Mr. Creager. W. P. Creager,† M. Am. Soc. C. E.—The author deserves a great deal of credit for his work in connection with these experiments. The curves showing equal-pressure contours under his model dams will be of great interest to engineers and of material assistance in connection with further studies on the subject.

The speaker feels the necessity, however, of calling attention to the danger of applying information gathered from experiments on a small scale to the design of structures of much greater magnitude. In order to indicate the grave discrepancies which may be expected

* The writer would refer, in this respect, to Buckley's "Irrigation Pocketbook", p. 362, *et seq.*, and, also, to Professional Memoirs, U. S. Engineer Corps, Vol. VII, p. 32, *et seq.*, especially to p. 44.

† New York City.

to result from the application, to large dams, of the author's formula for uplift, the speaker has computed from this formula the uplift on a dam with a 20-ft. base when subjected to a 30-ft. head of water. The author's formula follows: Mr. Creager.

$$P = 62.5\ bs + 62.5\ c\ (d^s - 1).$$

For this case, $H = 30$; $L = 20$; $s = 1.5$; $b = 278$; $c = 0.94$; and $d = 40.5$, from Table 6.

Therefore,

$$P = 62.5 \times 278 \times 1.5 + 62.5 \times 0.94\ (40.5^{1.5} - 1) = 41\,200.$$

The average uplift on the base is

$$p = \frac{P}{L} = \frac{41\ 200}{20} = 2\ 060 \text{ lb. per sq. ft.}$$

The average uplift head is $h = \dfrac{2\ 060}{62.5} = 33$ ft.

$$\frac{h}{H} = \frac{33}{30} = 1.10.$$

From which it is seen that, for this example, the author's formula indicates a 10% greater average uplift head on the base than the head on the dam. This, of course, is an impossibility, as the greatest uplift head which could possibly obtain would be exactly equal to the head on the dam, and the average uplift head could equal the head on the dam only if there was an impervious barrier at the toe of the dam.

The speaker has calculated from the author's formula and plotted on the diagram, Fig. 18, the average uplift head, in terms of head on the dam for a 20-ft. base and various values of s. He has also plotted two vertical lines representing the maximum "possible" uplift head and the "probable" uplift head for foundations of this type. The value, 0.5, for the speaker's probable uplift, was obtained by assuming an uplift head equal to H at the up-stream end of the base and diminishing uniformly to zero at the toe.

It is seen that the curve computed from the author's formula gives values of uplift at all points greater than the speaker's probable value, and for s larger than 1.4, it gives greater values than it is actually possible to obtain. Values of s greater than about 1.4 are not practical for dams with considerable uplift (unless built with a great superelevation above water surface) and this does not alter the fact that the author's formula for such conditions is incorrect. There is, then, some question as to the accuracy of the formula for lower values of s.

The direct results from the author's experiments are plotted in dotted lines on the diagram. They all give values of average uplift less than the speaker's probable value, which is a condition quite likely to occur.

Mr. Creager.

The speaker believes that the average uplift head on the base of an ordinary dam on a sand foundation without a cut-off is very close to $\frac{H}{2}$, irrespective of the actual size of the dam or the ratio of head to base length, provided the sand is equally pervious at all points. That the sand in the author's model was not of this nature is indicated by the variance of the contour intervals along the base of the dam (Fig. 4, etc.) and the fact that lines drawn through the actual points plotted on Fig. 11 would be anything but smooth. This probably accounts for the relatively large "entrance and exit" losses which the author mentions, but which the speaker does not attribute to "eddies".

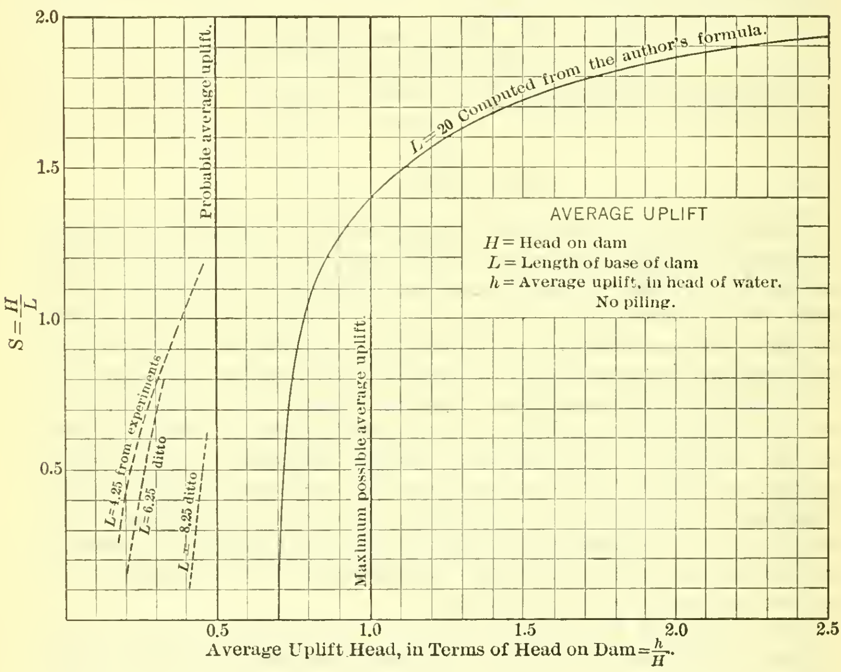


FIG. 18.

With reference to the effect of piling on uplift, the author found that "the 3-ft. piling gives total pressures * * * greater than those for the 1-ft. piling". As it is the speaker's opinion that the 5-ft. piling should have given less uplift than the 1-ft. piling, and as, if the piling had been driven to the bottom of the box, there would have been no uplift at all, the writer searched for an explanation of the author's statement and believes he has found the answer in Fig. 7. In the lowest diagram it will be noted that Pressure Contour 1.75 connects the bottom of the piling with the bottom of the box. Immediately

down stream from the top of the piling there appears a pressure contour, 2.00, and Piezometer Tube 46, also on the down-stream side of the piling, registered 2.45 (see Table 2). Mr Creager.

This condition could not have obtained unless the piling leaked badly at the top. This leakage evidently accounts for the fact that the author observed greater uplift for 5-ft. piling than for 1-ft. piling, the latter being tight, as shown by Fig. 5.

EDWARD WEGMANN,* M. AM. SOC. C. E.—The speaker must compliment the author and the University of Michigan on the scientific manner in which the experiments on the action of water under a model dam have been made and the results worked up. These experiments, however, were on a very small scale, and very different results may occur in the case of a dam having to sustain a pressure due to a large head. In these experiments, the greatest head was only about 5 ft. The base of the model dam was only 8 ft. 3 in., and the flow of the tail-water was affected by the end of the test box, which acted like a sheet-piling. The speaker does not consider it safe to apply to the case of a high dam some of the conclusions of the paper. Mr. Wegmann.

The author found that the porosity, effective size, and uniformity coefficients had little influence on the upward pressure exerted against the dam. The speaker thinks that these factors would have a very material effect on such pressure, except for a very low head, in which case viscosity of the water, capillary action, etc., might prevent the water from exerting much upward pressure.

As an instance of the difference between the effect of high and low heads, attention is called to experiments with the flow through orifices. With small heads and small orifices the coefficient of discharge is always greater than with large heads and large orifices. This is due to the viscosity of the water, capillary action, etc., which interferes with the contraction of the jet.

Something of this nature might interfere with the flow of water through sand, and prevent small heads from exerting much upward pressure; but, for high heads, these resistances have little or no effect.

The speaker cannot agree with the author's Conclusion 16:

"An impervious cut-off wall of comparatively shallow depth, say 5 ft., at the heel of the structure will greatly increase its stability by reducing the upward pressure on the floor."

This may be true for a 5-ft. head and a small base, but it is certainly not true for a head of about 50 ft. The failure of the dam at Austin, Pa., on September 30th, 1911, proved that conclusively. In this case, the cut-off wall had only been carried down about 6 ft. The water from the reservoir, under a head of about 45 ft., percolated

* New York City.

Mr. Wegmann.

under this wall, through sandstone and shale, and exerted an upward pressure which caused the failure of the dam. In the speaker's opinion, a shallow cut-off wall would have very little effect as regards upward pressure, and the only way to prevent such a pressure would be to carry down a well-built, water-tight cut-off to an impervious stratum.

Recently the speaker has had some interesting experiences relative to the manner in which pressure may be transmitted under the base of a dam. In the case he has in mind, an earthen dam, about 25 ft. high, had to be constructed for about 250 ft. of its length on soft, marshy ground, underlain at the depth of 27 ft. by sand. The dam was 10 ft. wide on top. The up-stream side was sloped 2 to 1, but the slope was broken 14 ft. above the base by a berm 5 ft. wide. On the down-stream side a similar berm, 6½ ft. wide, was placed about 12 ft. above the base. Below this berm, the down-stream slope was made 2 to 1, but above it, the slope was only 1½ to 1. A core-wall, founded on a pile foundation, was constructed in the center of the earthen dam, and, in order to prevent the water from leaking under the dam, tight sheet-piling was driven on the up-stream side of the pile foundation.

The dam would have stood successfully, if it had been founded on hard ground, but, for the marsh that had to be crossed, the plan adopted had the great defect of placing a considerably greater pressure on one side of the core-wall than on the other. In the former case the pressure was caused by the water and the saturated earth on the up-stream side of the core-wall. In the latter case the pressure was due to the dry earth on the down-stream side of the core-wall. The difference in pressure was transmitted through the marshy ground almost like the hydrostatic pressure of water, and forced the foundation platform of the core-wall and the sheet-piling down stream. The whole core-wall followed the same movement, and, when the speaker was consulted about what remedies should be adopted, the top of the core-wall had moved more than 3 ft. down stream.

A simple calculation showed the inequality of the forces acting on the two sides of the core-wall, and the speaker recommended an increase of 15 ft. in the top width of the dam and making the down-stream slope 3 to 1. These measures were carried out, and no further trouble has been experienced.

In the speaker's opinion, the proper way to have crossed the marsh would have been by constructing a homogeneous earth dike, having a greater slope on the down-stream than on the up-stream side, so as to offset the weight of water on the up-stream slope. The dam was bound to settle, and the only remedy for this was to continue filling until the soft material under it had been sufficiently compressed to bear the weight of the embankment.

Mr. Wegmann.

If sheet-piling had been required to prevent leakage under the dam, it should have been driven, either at the toe of the up-stream slope, or on the center line of the dam, after the material under it had reached a state of equilibrium.

The speaker had a similar experience on the New York, West Shore and Buffalo Railroad, which showed how pressure was transmitted through soft ground. In this case two abutments, each 45 ft. high, had to be built for a 60-ft. plate-girder bridge which spanned Cedar Pond Creek, near West Haverstraw, N. Y. The ground at the site of the bridge was very similar to that on which the dam just mentioned had to be constructed. A long embankment, 45 ft. high, carried the railroad from the south side of the valley to the south embankment. A foundation was prepared for this embankment by corduroying the ground with two courses of round timber. On the north side of the bridge, only a short embankment was required. Both abutments were built on pile foundations.

When the toe of the south bank had about reached the south abutment, the weight of the embankment, although well distributed by the corduroy timbers, caused a pressure on the soft ground, which forced the piling toward the creek. A similar movement occurred in the north abutment. The displacement of the two abutments, which reduced the original span by about 1½ ft., was due partly to scouring in the creek bed, which increased the depth of water from about 2 to 6 ft.

The moving of the abutments was stopped temporarily by bracing one against the other by 60-ft. piles, bolted together in pairs, tip to butt. A permanent bracing of 12 by 12-in. timbers, looking like an inverted roof, was then placed between the abutments, the peak resting on a crib sunk half way between them. Further scouring was prevented by driving sheet-piling above and below the abutments. The space between the two rows of sheet-piles was filled with stones. After the work had been completed, the temporary bracing was removed.

No further trouble was experienced with the abutments, and for many years they were held safely by the bracing described. The speaker understands that some changes or repairs have recently been made to the abutments, but he does not know their nature.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE DESIGN OF HYDRO-ELECTRIC POWER PLANTS

Discussion.*

By J. D. Galloway, M. Am. Soc. C. E.†

J. D. Galloway,‡ M. Am. Soc. C. E. (by letter).—Several who have discussed the paper have referred to the subject of load factor. It is well known that a load factor can be built up by good management, and Mr. Newman has noted how that of the San Joaquin Light and Power Company was increased from a relatively low one to one of 76%, which is relatively high. Mr. Homberger has also remarked on the increase of load factor. In earlier times, this could only be guessed at, but it would seem that, with the experience of many companies available, a much closer approximation to the probable load factor could be made. An engineer with Mr. Newman's years of experience could now go into new territory and make a fair estimate of a possible load factor. Mr. Galloway.

Mr. Homberger questions the statement concerning the given load factor of the Sierra and San Francisco Power Company. The data of the street-car system of San Francisco were given to the writer by the General Manager, and are believed to be correct.

Mr. Pfau has expressed a belief that where a kilowatt-hour produced by steam costs more than one produced by water-power, the steam units should be allowed to operate continuously at or near the most efficient point, the load variations being taken by the water-power plant. The writer is not prepared to say that this is not true in some cases, possibly where the two plants are near each other. However, when the transmission line is long, it is believed that the steam plant should only

* Discussion of the paper by J. D. Galloway, M. Am. Soc. C. E., continued from September, 1915, *Proceedings*.

† Author's closure.

‡ San Francisco, Cal.

Mr. Galloway.

be used to take the peak and the water plant allowed to produce the major part of the energy. The investment in plant is much reduced, as noted in the paper, to which the reader is referred.

Mr. Newman makes some valuable comments on the value of n in Kutter's formula, and cites data based on experience. His allowance for contingencies in the design of conduits is quite to the point. Most engineers will add something to the size of a conduit after designing it by formula. As to the design of the regulating reservoir, Mr. Newman overlooked the writer's statement that, although the reservoir capacity can be obtained by integration from the load curve, it should at least hold water for one day's run or more, if circumstances will permit.

The number of units in a plant was commented on by Mr. Homberger and Mr. Newman. The latter overlooked the writer's statement that the small plant with three units is an isolated plant. It was noted in the paper that when such a plant is part of a large system, one unit is sufficient, and the Deer Creek Plant of the Pacific Gas and Electric Corporation was cited as an example. Mr. Homberger states that the general rules given as to the number of units are not borne out by the design of plants in the United States or in Europe. The writer referred to modern plants. Earlier plants, which were subject to the changes of evolution, are not examples. Such plants as the Drum Plant or the two Big Creek Plants, in California, both recently constructed, are good examples. In both, the number of generators was kept low and the size was increased above those formerly used. At the Drum Plant four 12 500-kw. units form the ultimate installation. If the plant had been one of an isolated system, possibly the designer would have added another as a spare unit. Under the conditions, the plant being part of a large system, four very large units were advisable. Mr. Homberger would hardly advise the use of ten 6 000-kw. units, or some other similar combination, for this plant.

The writer's reasons for the statements regarding the number of pipe lines are as follows: Two pipes are to be preferred to one. If an accident happened to the single pipe, the entire plant would have to be closed down during repairs. Two pipes will allow one to operate nearly all the plant during repairs to the other. Not more than two units on one pipe were advised, for the reason that an injury to the pipe would shut down too much generating capacity. Individual pipes to each generator in low and medium-head plants were advised because the quantity of water is great, and cross-connections, elbows, or Y's are difficult to construct. The theoretical quantity of metal in one large pipe is equal to that in two smaller pipes of the same total capacity.

The writer differs from Mr. Creager in the assumption as to the pressure in a pipe from surges varying uniformly from a maximum

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at the power-house to zero at the upper end. Experience at two power stations shows that the increase of pressure near the top of the pipe is almost the same as that at the bottom, as recorded by gauges. The writer believes that it is not well to design a water-wheel operating under a head of 2 000 ft., the closing of the gates of which may cause an increase in pressure over that of the static head equal to 100 per cent. It would be a waste of money in the pipe and also a dangerous condition in the unit. Such a unit would require an impulse-wheel. If it is provided with a deflecting nozzle or proper by-pass, the water column is not checked by governor changes, and little or no pressure rise results. Relief valves on turbines are not quite as efficient, but modern ones are built, which keep the surge pressures far below the quantity cited.

Mr. Homberger questions the statement that during a great part of the year the efficiency of the water-wheel is not important. It was not intended as a reason for buying a machine with an efficiency of 80%, when one of 90% was available. The basis of the argument can be illustrated by an example: Assume a 10 000-kw. generator, one of a number in a power-house. A difference in efficiency of 1% represents 100 kw., equivalent to 876 000 kw-hr. per year, which, at a value of ½ cent per kilowatt-hour, represents a difference of $4 380. Capitalized at 6%, this represents $73 000 as the value of the 1 per cent. The amount is greater than the total cost of the hydraulic unit. Obviously, the purchaser could not pay that much additional for the 1%, for the reason that, during a great part of the year, no returns could be had on the extra 1 per cent. A little more water through the less efficient wheel will produce the maximum output from the generator. Throughout most of the year, this water is available and, if not used, goes over the dam and is wasted. Such considerations reduce materially the value of the saving of a small percentage in the efficiency of wheels. However, this in no way should lead to the selection of a low-efficiency unit.

The remarks, by several of those engaged in the manufacture of water-wheels, as to the limiting values of the specific speed are to be taken as authoritative. The writer was not in a position to offer any but general data on this subject, based on given installations. The paper mentioned by Mr. Pfau, which is to be presented before the International Engineering Congress, 1915, will be a considerable addition to the literature on this subject. The extreme limits of specific speed given for some installations cannot, however, indicate ordinary limits of good design. This is given rather by the general formula or by definite statements.

The writer desires to express his appreciation of the kindness of those who have taken the trouble to discuss the paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

ADDRESS AT THE ANNUAL CONVENTION, IN SAN FRANCISCO, CAL., SEPTEMBER 16TH, 1915

IDEALISM AND ART IN ENGINEERING

By Charles D. Marx, President, Am. Soc. C. E.

The brook of scientific and industrial development, so tiny at the beginning of the last century, has steadily grown deeper and stronger; new tributaries—the developments of scientific and industrial specialties—have added new volumes, until at the beginning of this new century we find it a mighty stream, deep with thought, broad with liberality, and symbolical of modern scientific and industrial development, flowing in the direction of human progress.

This stream, which scientists have done much to strengthen and direct, it is said, has washed away the old landmarks of idealism and art. Sad, indeed, if this were true, but I think the foundations of both are laid too deep to be scoured out even by the flood of which I have spoken. Let us examine dispassionately, and as thoroughly as is consistent with brevity, the serious charges which many still bring against science, pure and applied.

What determines the existence of ideals in the life of an individual—in that of a nation? The man of the highest ideals is not a man of words, but a man of action. He is not content, as Dean A. W. Smith of Sibley College said in his beautiful tribute to Ex-President A. D. White, to be a mere seer of visions.

> "Some men who see rare visions rest content
> To see them and to let them fade away;
> Not so with him; to him the vision meant
> The call to toil to make the vision stay."

Numberless are the instances in the past as well as in the present where poets have sung and clergymen preached self-denial, self-sacrifice, humility and service, the essentials of an ideal life as typified in Christ. Numberless too, I am sorry to say, are the instances in which the avowal of such ideals has not carried in its wake a living up to them. The possession of ideals, or rather the being possessed by them, must show itself in a man's life, and is in a large measure independent of his occupation. I am fully aware that in making this statement I am running counter to Mr. Ruskin, for he inveighs strongly against what he calls the "thought-killing work of the masses of our laboring classes", and for a good deal of this work the Profession of Engineering is responsible. Though not blind to the fact that ten hours a day of incessant toil in digging a trench or running a machine will not develop a man as we should like to see him developed, I still claim that if in such a case the ideals in a man's breast are killed, it is owing to the amount of work and not to its character. Physical exhaustion has become so great, that mental exertion is out of the question, and man is brought to the level of the brute. But even under these most unfavorable conditions we have numberless instances of men and women leading ideal lives—heroes and heroines living and dying unsung. Now and then a brief notice, such as I cut out of one of our large dailies, passes under your eye:

"Napoleon de Montague, a miner, was killed yesterday in Lance colliery in Plymouth, while endeavoring to save his fellow-workmen from a terrible death. He had fired a shot and ran behind a pillar, when the shot exploded, and the flash set fire to some gas near the roof of the chamber communicating with the main gangway. Realizing that the fire might spread in a moment through the whole mine, de Montague tore off his coat and smothered the flame. Just as the fire was extinguished the roof of the chamber, loosened by the blast, fell upon and killed him. His act probably saved many lives."

You give this notice but a passing glance. It lacks the glamor of romanticism. How much nobler, how much more ideal it would seem to our romantic friend, if it were the deed of valor of a knight errant who fell defending some imaginary slight cast upon some imaginary honor. This, however, is only a case of self-sacrifice in the utilitarian occupation of coal mining, illy suited for poetic garb and hardly fit for polite society. How can idealism and utilitarianism mate? We

all travel unhesitatingly on land and on water. We confide our bodies, which we generally deem of more value than our souls, to car or boat. Have you ever fully realized, in reading accounts of the comparatively few accidents that do occur, that most of them are accompanied by some such statement as: "the engineer reversed his lever, stuck to his post, and was found dead under his engine", or "the pilot stood at the helm until the burning boat was beached"? These cases are so numerous that it seems a platitude to speak of them. Acts of heroism as great as any that were done in the past, as noble as any, for they are performed, not in the much sung vocation of "killing" our neighbor, but in that "despised" one of "saving" him. In a recent number of *The Forum,* H. M. Chittenden, Brigadier General, U. S. Army (Retired) and an honored member of society, has an article on "Peace and Heroism" the reading of which I recommend most heartily. It is a splendid tribute to the heroes of civil life. I really should quote the article as a whole, did space permit, but I must confine myself to a brief extract:

"But while war, in the very nature of things, abounds in opportunities for valorous exploits, and its every deed is written large on the page of history, the humbler and quieter sphere of private life affords even more and keener opportunities for the display of true heroism. The physician or nurse, who voluntarily goes into a plague-stricken district, the miner who braves the fire-damp to rescue his imprisoned fellows, the crew who stand at their posts while their vessel is sinking, the fireman who scales a tottering wall to save a human life, the patrolman who enters a den of desperadoes at imminent personal risk—whoever in the pursuance of duty, no matter how humble, subordinates his personal safety to that duty—is as much entitled to the commendation of heroism as a soldier who does his duty in war can possibly be. These opportunities for heroic deeds are everywhere with us and always will be. They may lack the glamor of war and go unblazoned to the world, but the very humbleness of their status enhances, if anything, their heroic quality."

And as Dr. Jordan has well put it, in speaking of the work of a member of this Society, who has had charge of the relief of six millions of Belgians against "democratic famine working day and night", "this was a problem of infinite dimensions". But an engineer, "born with a streak of idealism" and with great executive ability, tackled this problem and solved it. Solved it so well that one of his colleagues in the work writes: "It's often been said that our American Commis-

sion is the hope of Belgium. Revise that—it's Hoover, that's the hope of Belgium."

Engineering destructive of idealism! Little study has he made, who advances this claim, of the close association between religion and engineering in the past—between religion and engineering in the present. The monks of the Twelfth Century who toiled patiently placing stone on stone in the bridges which they erected, saw more clearly than do many of the present century the relation between paths of communication and the spread of religion. They saw and felt that the greater the number of timber and stone ties with which they bound adjacent countries together, the greater would become the number of spiritual ties, the more would recede the causes of war, the nearer would approach the day of "on earth peace, good will toward men". And toward this same end the men in the Profession of Engineering have been laboring. It is true that at the present time, these ties, which we have been so long in constructing are being severed, but for the severing of these ties the Profession of Engineering is not responsible.

Now let us turn in detail to various branches of our Profession, and see if the practice of them is likely to be destructive of idealism. The Mosaic code of laws shows that the health of the Jewish population was a matter of such supreme interest that special legislation on that subject seemed warranted. The Greeks and particularly the Romans recognized fully the moral value of personal cleanliness; but it has been left to our country to show clearly and on the basis of scientific study, the interdependence between sanitation and disease, sewerage and crime. Münsterberg has said, "Hygiene can prevent more crime than any law". Speaking before the Massachusetts Conference of Health Officials, Dr. Charles W. Eliot, President Emeritus of Harvard, addressed them in part as follows:

"The progress in knowledge of preventive medicine made during the past fifty years, and in applications of that knowledge in social practice, has been the most cheerful phenomenon in the recent history of civilization. The new applications of physical forces—heat, light and electricity, which mankind has learned to use in its conflict with nature—have proved to be highly beneficent in the field of preventive medicine. Civilized communities have been enabled to make their water supplies, food supplies and drainage systems safe, and to contend with

unexampled success against formidable pestilences, the common communicable diseases, and the bodily ills which attend urban life and the factory system."

In this work of preventive medicine the sanitary engineer has borne his full share. As we read these inspiring words of Dr. Eliot our thoughts turn at once to that monumental work in sanitation carried out on the Panama Canal under the direction of General Gorgas, and with the hearty co-operation of the engineering staff. Under disheartening difficulties, and with sacrifice of personal comfort, yes, at the risk of their lives, the men labored, who made the Isthmus a place fit for the white man to live, and thereby made the construction of the canal, which means so much to mankind, possible. Honor, deserved honor, has come to him who directed this splendid piece of sanitary engineering, but I desire to put on record here the appreciation of this Society for the men who stood by. Is it reasonable to suppose that the engineers who did this work, who made these sacrifices, were not the highest type of men, *i. e.,* practical idealists? I could go on and specify instance on instance in which modern sanitary engineering has achieved results similar to those already brought before you. Most of you have no doubt read Dr. Shaw's extremely interesting papers on municipal reforms in London, Glasgow, Berlin, Paris, Vienna, Naples, and even ancient Rome. That these last named cities should also be attacked by the modern spirit of reform seems to our romantic friends proof conclusive that "ideals" have perished. We are called iconoclast engineers, utilitarians, non-respectors of tradition, antiquity, and picturesqueness. Iconoclasts, yes; we plead guilty to the charge; reformers usually are. Utilitarians; that too is an accusation that we cannot deny; but is an act less noble for being useful? Non-respecters of tradition and antiquity! Again we plead guilty, if tradition stands for error, and antiquity for decay. Destroyers of the picturesque; a general denial cannot be entered against this charge either; and in some cases, I am free to confess, too little justification can be shown for the disregard of the picturesque. This is a point I shall touch upon more fully later; but with special application to the cases cited above and similar ones, what is picturesqueness but the diseased condition of structures and their surroundings? What is this vaunted love of the picturesque in many cases, but a selfish and thoughtless appreciation of surface appearances? Selfish and thoughtless, I say, because the few

are willing to sacrifice the weal and woe, the health and happiness of the many, in order that they may feast their eyes on narrow winding streets, on quaint gables, far-reaching eaves, small, curiously leaded panes of glass. They are taken in by the surface appearance of things. But little heed give they to the squalor and dirt, the misery and sickness existing in these picturesque quarters. If but sunshine stream merrily through the broad, light panes in their houses, if but their sanitary appliances are of the best, what thought give they to the life of those who are huddled into those picturesque quarters? Fondly their thoughts turn back to the heroes of the past, to them their hearts go out. The men of the present become heroes to them only as they too become a part of the past. Let me ask you: Who then is the idealist? The man who, probing into the sore which has so little surface indication, finds its deep-rooted seat, and skilfully uses the knife; or the man who, misled by these same facts, applies a surface dressing and allows the sore to eat into the body? To the thinking man the answer is simple. Perhaps you will grant now that one branch of engineering, at least, namely, sanitary engineering, and idealism, are not only not incompatible, but that they are almost inseparable. And what I have shown somewhat fully for this branch of the Profession, can be shown as well for the many other branches.

Take irrigation engineering, for instance, the possibilities of which are only beginning to be realized in this country. The massive dams now building in the fastnesses of the Sierras, the Rockies, and the other mountain ranges from which spring our rivers, will store safely behind their broad backs the precious water which has long run to waste. Thousands of miles of ditches, of pipes in iron, steel, and wood, will lead this water to the thirsty soil. The wonders which Mother Earth, in gratitude for this quenching of her thirst, accomplished are, to many of you, wonders no longer. Is it likely, I again ask, that the men carrying out these works see in them but the piling of one stone upon another, the digging of so many feet of trench, the laying of so many feet of pipe? Believe me, these black cast- or wrought-iron cylinders stand for more than this to the true engineer. He realizes that with every water or drain pipe well laid he is bringing prosperity and happiness, health and vigor, where before existed poverty and misery, sickness and languor. Perhaps the most wonderful instance on record in modern times of the far-reaching effects of irrigation engineering is

found in Egypt. In an article on the regeneration of Egypt, by the former librarian at Stanford, Mr. Woodruff (now Professor of Law at Cornell), he unhesitatingly and justly, I think, attributes a large share of the credit for this "new birth" to the work of the English engineers. Mr. Woodruff says:

"The history of the English in the administration of Egypt for the past nine years is the record of the return to health, strength and prosperity of a country that has been bled and starved almost beyond resuscitation. And yet there has been little romance in this restoration. It is chiefly a story of common sense, honesty and straightforward hard work."

Mr. Woodruff then states more in detail the works of construction and repair carried out by the English engineers and pays the following tribute to the latter as men of high ideals:

"A word must be said as to the character of these engineers who have been foremost in the redemption of Egypt. They have had to contend with vested abuses on every side, learn the spoken Arabic of the common people and overcome religious prejudices and superstitions which, as Balzac says, are the most indestructible form of human thought. Each of the present irrigation inspectors travels his district again and again, often on foot, suffering much hardship and seeking the shelter of the humblest mud-huts lest by accepting the entertainment of the wealthy proprietors he be suspected by the poorer natives of having been bribed. In former days a poor man was completely at the mercy of his rich neighbor and of the corrupt native inspector, who unless bribed, would not open a sluice at a critical time for the crops. Now the poor no longer have to bribe for water, they have confidence in the English inspectors and have learned that petitions will be listened to and wrongs redressed."

And to this strong tribute Viscount Milner in his book on "England and Egypt", has added even a stronger one in his chapter on "The Struggle for Water". I have never read a more interesting or appreciative description of the work of the engineer. I can but recommend to you the reading of it, and must content myself with but a brief extract from this chapter, and the retelling of a story which shows the appreciation in which our brother engineers are held. First Viscount Milner says:

"The longer I remained in Egypt, and the more I saw of the country, the more clear it became to me that the work of these men has been the basis of all the material improvement of the past ten years."

Then he continues:

"Only one case in point to conclude with. It is a story which I think I have seen in print before, but it is so remarkable that it will, perhaps, bear repetition. In the bad year 1888, when, as has been stated, the Nile flood was an exceptionally poor one, there was a large area in the province of Girga which was threatened, like many others in Upper Egypt, with a total failure of the inundation. The canal which ordinarily flooded this particular district was running at a level at which the water could not possibly spread over the fields, and many thousands of acres seemed doomed to absolute barrenness. A cry of despair arose from the whole neighborhood—What was to be done? One of the English Inspectors of Irrigation, who happened to be on the spot, promptly determined to throw a temporary dam across the Canal. The idea was a bold one. The time was short. The Canal was large, and, though lower than usual, it was still carrying a great body of water at a considerable velocity. Of course, no preparations had been made for a work the necessity of which had never been contemplated; but the Inspector was not to be daunted by the apparent hopelessness of the undertaking. Labour, at any rate, was forthcoming in any quantity, for the people, who saw starvation staring them in the face, needed no compulsion to join gladly in any enterprize which offered them even the remotest chance of relief. So the Inspector hastily got together the best material within reach. He brought his bed on to the Canal bank, and did not leave the scene of operations, night or day, till the work was finished. And the plan succeeded. To the surprise of all, the dam was, somehow or other, made strong enough to resist the current. The water was raised to the required level, and the land was effectually flooded.

"The joy and the gratitude of the people knew no bounds. It was decided to offer thanksgiving in the Mosque of the chief town of the district, and the event was considered of such general importance that the Minister of Public Works himself made a special point of attending the ceremony. But the enthusiastic population were not content with the presence of the high native dignitary. They insisted that his English subordinate also should be there. They were not willing to give thanks for their deliverance without having amongst them the man who had wrought it. Every one knows how deep a prejudice exists in Mohammedan countries against the presence of a Christian in a mosque. In the great tourist-visited cities of Egypt this feeling is wearing off, but in the country districts it is as strong as ever. In those districts it is an unheard-of thing that a Christian should be present at a religious ceremony—more than unheard-of that he should be present at the instance of the Mohammedan worshippers themselves. But in this case the universal feeling of thankfulness and admiration was too strong for the most deeply-rooted fanaticism. For the first

time, doubtless, in the history of that neighborhood, an Englishman and a Christian was allowed, and even compelled, by the natives, to take part in a solemn function of their usually exclusive and intolerant faith."

But we need not turn to foreign countries to find work of the irrigation engineer worthy of commendation. Our own Reclamation Service has made a record of which we may well feel proud. Here too the men, with a devotion to the cause of mankind, with loyalty and steadfastness, have given a service, the value of which is not yet fully recognized. These are the men of whom Senator Newlands said, in the United States Senate, on February 17th, 1910:

"We have had one of the most capable and honest construction services organized that has ever existed in the history of this country. The Committee of the Senate on Irrigation has been engaged during the past year in visiting these various works, and not a whisper of corruption has reached them. It has been a work conducted with rare integrity and with rare speed. In carrying out the Egyptian irrigation projects, both works and settlers were financed. If our projects are to rival them in success, our Government must adopt that policy. The engineers have done their work well; they are the men of whom Chief Engineer Davis said: 'Their chief tie to the service is not the matter of salary, but interest in their work and loyalty to it and the belief that they are appreciated'."

Again, I ask, is such work destructive of idealism, are such men lacking in ideals?

In railroad engineering, think you that the men who through virgin forests and sandy deserts, through miasmatic swamps and rocky cañon, across rivers and over mountains, carried the steel bands that now tie mankind so closely together—think you that these men were engaged in an occupation likely to kill their ideals? When the final balance is struck, I warrant that the debit will not be on the side of this grand army of peace of the present, as compared with the armies of war of the past and present, for deeds of ideal heroism, self-sacrifice, and devotion to duty. It seems like "carrying coals to Newcastle" to speak in an audience like this of what the railroads have done for all countries—for our own country especially, and more particularly for the Pacific slope. It was not so long ago when I read of the beginning of construction of the Trans-Siberian Railway which now unites the Atlantic and the Pacific on the other continent. The Cape to Cairo Railway, too, has passed through the stages of its preliminary surveys

and partial construction. What centuries of fighting could not accomplish, these two roads will in time accomplish. The light of civilization will be spread on the Dark Continent, and its strong rays will burst the fetters and open the prison doors of suffering men and women in Russia. Read the interesting description of what has already been done in the Sahara, in one of the recent numbers of *Scribner's Magazine*. The men who, like M. Rolland, have made the desert to "blossom as the rose", are they likely to see no further than the mere short span of the present? Does not their imagination people these stretches of land with busy towns and bustling marts, with peaceful villages and happy homes? Is it destructive of ideals to have contributed, if ever so slightly, to the realization of such a dream? True, the railroad has penetrated into the haunts and sacred preserves of our romantic friends—the wonderful majesty of the Alps, the somber beauty of the Black Forest, the towering ruggedness and inspiring beauty of our own Rockies, have become the possession of the many instead of remaining the privilege of the few. Nature has become vulgarized complains the romanticist, because for the many whose faith has been strengthened, nay created, as they view the wonders of Nature spread out before them, there may be shown a few whose souls are not attuned in sympathy with their surroundings—a few who in the midst of such surroundings cling to what is "of the earth earthy". Dr. Waldstein aptly says, in an admirable article on the works of Ruskin:

"There can be no doubt that our enjoyment must be impaired by the reduction of what stimulates our higher ambitions to a common place; but we must willingly make this sacrifice when we consider the great gain accruing to hundreds or thousands, where before it but reached units."

Who, then, is destructive of idealism? The man whose works are a means, if but a humble one, of bringing his fellow-beings into a direct contact with the wonders of creation; or he who, enveloped in the mantle of exclusiveness, bemoans this defiling contact?

Now let us turn, if but briefly, to bridge engineering. Select from the many wonderful structures of the present century but one. To some a bridge is merely a means of safely transferring men and goods from one side of a valley, or one bank of a river, to the other. It may stand for much more to others. Take the famous East River Bridge.

Thousands throng it daily, on foot and by rail. Bright and cheerful, some cross in the morning but to return despondent and cheerless at night. Hopes deferred—hopes realized—obstacles overcome—overcome by obstacles: and so the stream of humanity flows across it day by day. Is that bridge merely so many pounds of iron, steel, and wood, suspended in mid air according to the laws of the strength of materials? Is that bridge but the embodiment of cold mathematical calculations? Surely it meant more to the noble engineer whose life was sacrificed in the building, to the devoted wife who added her patient hours of toil to those of her husband that he might not die before he had finished his great task. Was that a life devoid of ideals? Was that a work destructive of idealism?

What bridge engineer has not been touched by the romance of his Profession? What engineer has not been thrilled by Kipling's story of the "Bridge Builders" where that masterhand has voiced, in prose which is almost poetry, the thoughts that came to the engineer as he viewed his work:

"It was a long, long reverie, and it covered storm, sudden freshets, death in every manner and shape, violent and awful rage against red tape half frenzying a mind that knows it should be busy on other things; drought, sanitation, finance; birth, wedding, burial, and riot in the village of twenty warring castes; argument, expostulation, persuasion, and the blank despair that a man goes to bed upon, thankful that his rifle is all in pieces in the gun-case. Behind everything rose the black frame of the Kashi bridge—plate by plate, girder by girder, span by span—and each piece of it recalled Hitchcock, the all-round man, who had stood by his chief without failing him from the very first to this last."

If I desire to show that in still another branch of engineering—the one of river and harbor improvement—there is nothing destructive of idealism, I need not go far for an illustrious example. What James B. Eads has done for the people living in the Mississippi Valley in opening up the mouth of the Father of Waters, stamps him as one of the benefactors of mankind. For years he labored and fought, removed mountains of obstacles, overcame prejudice, malice and ignorance. Mr. Corthell, the staunch friend and principal assistant of Captain Eads on this momentous work, quotes him as saying:

"I therefore undertake the work with a faith based upon the ever constant ordinances of God himself; and so certainly as He will spare

my life and faculties for two years more, I will give to the Mississippi, through His Grace and by the application of His laws, a deep, open, safe, and permanent outlet to the seas."

Tireless energy, self-sacrifice, devotion to the welfare of mankind, make up a life as noble as any that was ever lived. Nor was this man bereft of ideals, who, when told, in a foreign country, that he was dying and could live but little longer, said: "I cannot die; I have not finished my work!"

Need I add to this a reference to that stupendous piece of engineering, which looms large in the memories of us all to-day. General Goethals, his predecessors and associates, have given the world an example of the earnestness, efficiency, and devotion which animates the members of our Profession. In the article previously quoted, General Chittenden says:

"As an example of national heroism—the making of a great sacrifice to accomplish a worthy purpose—it may rank with the most righteous wars."

May I be forgiven for mentioning by name one who contributed so much to the success of this enterprise, and whose name now is closely linked with that part of the work for which he sacrificed his life. Of him his chief has said:

"Colonel Gaillard was a great engineer and unflinching worker and a true gentleman. Gaillard Cut is a worthy monument to his name."

It is characteristic of George Eliot, that, with her marvelous insight into human nature, with that clear understanding which comes only of broad sympathies, she should have drawn that humble but lovely character of Caleb Garth—a type of the technical idealist. Do you recall the passage in which she says:

"Caleb Garth often shook his head in meditation on the value, the indispensable might of that myriad-headed, myriad-handed labor, by which the social body is fed, clothed and housed. It had laid hold of his imagination in boyhood. The echoes of the great hammer where roof or keel were amaking, the signal shouts of the workmen, the roar of the furnace, the thunder and plash of the engine, were a sublime music to him; the felling and the lading of timber, and the huge trunk vibrating star-like in the distance along the highway, the crane at work on the wharf, the piled-up produce in warehouses, the precision and variety of muscular effort wherever exact work had to

be turned out—all these sights of his youth had acted upon him as poetry, without the aid of the poets, had made a philosophy for him without the aid of philosophers, a religion without the aid of theology."

I think enough has been brought before you now to show clearly that engineering is not destructive of idealism. That much refuted, there still remains the charge that engineering is destructive of, or at least in part responsible for the decay of, art. I propose to show that this statement also is false. Artist and romanticist appear as accusers. Again they point backward and say: "See what the past has created; what have we that can be placed by its side?" Their eyes are blinded to the changed condition of things. They lack the sympathetic understanding of the complex problems of the life of to-day, and the materialized solution of these problems does not appeal to their idea of the beautiful. For the intelligent enjoyment, and more particularly for the criticism, of any creation, there is needed at least a fair knowledge of the underlying principles of construction, be that a work of symphony, a poem, or a bridge. It is true that a symphony or a poem appeals much more readily to a large audience than does a bridge or a complicated piece of machinery; yet both the latter may be as much works of art as the former, a higher degree of development of the intellect being needed, however, to see and feel their beauty. Every engineering structure is the materialized idea of its function. This first step in its construction gives us the core, the mere form, if you will; but, as I believe I have amply shown, the ideas underlying engineering works are often ideal ones, and the works themselves, therefore, can be idealized. When this is done, the engineering structure becomes a work of art. Its form may not at once strike us as beautiful, and keen discipline may have to be ours before we can see its beauties. But is that reason sufficient to condemn it? Ask the admirers of Browning, Whitman, or Wagner. It would seem, therefore, as though our idea of the beautiful is not a fixed one. In all ages that which most truthfully and characteristically embodied in itself the representation of the life and the ideas of those times was deemed a work of art. He is the artist who expresses most faithfully what we think and feel. If such representation has not been had in our century, it is not for lack of new ideas and materials furnished by science pure and applied, but for the lack of adequate assimilating power on the part of the would-be apostles of the beautiful. The cry

against machine-made ornament, machine-made reproductions of works of art, is justified, if it is directed merely against the untruths that often accompany such reproductions—if it is directed against the attempt to make material seem other than it is. But the cry becomes senseless when directed against those processes which place good, honest reproductions of beautiful works in the possession of the many, where once they were but the property of the few. Art will not become vulgarized if a good copy of the Sistine Madonna is found in every household, or a copy of the statue of the Venus of Milo in every town. One's love of the beautiful grows by being surrounded by beautiful things, and need there be any further justification than this for the plea of putting the imprint of beauty on the surroundings of our daily lives? Yet there are those who deny the claim to engineering structures. Art romanticists deny the possibility; strict utilitarians, the desirability; capitalists, the rentability. The possibility has been shown; witness the marvelous palaces from which radiate the myriad lines that bind us together in common activity, common enjoyment. Structures that mark an epoch, not worthy of being beautiful! Is it true, as Ruskin says: "That there never was more flagrant nor impertinent folly than the smallest portion of ornament concerned with railways or near them"? Is criticism such as this likely to prove as beneficial to the creation of works of art as is the work of men who are fully alive to the needs of the present, in sympathy with humanity of to-day, and desirous of giving that fact its noblest expression in their works?

Turn now, briefly, to the desirability of beautifying engineering structures. Our opponents become more dangerous because they number on their side some of the members of the Engineering Profession. All art is luxury, they hold, and unworthy of thinking man; hence engineering structures, which are largely the product of thought based on mathematical unalterable conclusions, should be left in all the nakedness of the constructive form. The beauty is there, they say, if you can but see it. And they are right in part, but, as Professor Lucae well says, "It is not a finished art form you are dealing with any more than is the human body with its exposed muscles and ligatures." The idealizing of the materialized idea of the function of the structure, therefore, surely seems desirable.

And right here it is encouraging to note that this desirability was recognized by our own Government in connection with the structures of the Panama Canal, when it sent the sculptor, Daniel C. French, and the landscape architect, Frederick Law Olmsted, to report on the artistic character of the Panama Canal structures, and to make suggestions for such improvements as to them seemed desirable.

"The canal itself and all the structures connected with it impress one with a sense of their having been built with a view strictly to their utility. There is an entire absence of ornament and no evidence that the aesthetic has been considered except in a few cases as a secondary consideration. Because of this very fact there is little to find fault with from the artist's point of view. The canal, like the Pyramids or some imposing object in natural scenery, is impressive from its scale and simplicity and directness. One feels that anything done merely for the purpose of beautifying it would not only fail to accomplish the purpose, but would be an impertinence."

Thus spoke the true artists.

Is it profitable to beautify engineering structures? Here we stand before a momentous question. If the answer be given by the engineer, or by one who holds that the status of a people is determined not merely by the accumulated wealth of the Nation, the quantity of goods produced, and of articles manufactured, then it will be in the positive, ten times over; but if the man of low ideals and mercenary motives gives answer, it is likely to be an emphatic "No". This answer has been given too often in our own country, and the blame for the deep scars in the face of Nature, the ugly dams and rugged cuts, must not be laid on the shoulders of the engineer, who fain would heal with loving hand, and protecting sward the wounds he has struck. Where broad-minded liberality and far-seeing policy govern the construction of engineering works, as is the case in countries older than our own, these works stand as worthy art products of the spirit of the times symbolical of the best and highest in the life of to-day.

Science and its applied form, Engineering, therefore, have not been destructive of idealism, for, in the words of another: "When the period of history we now call modern will be rounded to completeness, all the highest and most sacred human ideals will not be lost or dimmed, but will become nearer and more real," and science has not been destructive of art or beauty. As Emerson says:

"Beauty will not come at the call of a legislature, nor will it repeat in England or America its history in Greece. It will come, as always, unannounced, and spring up between the feet of brave and earnest men. It is in vain that we look for genius to reiterate its marvels in the old arts; it is its instinct to find beauty and holiness in new and necessary facts, in the field and road-side, in the shop and mill. Proceeding from a religious heart, it will raise to a divine use the railroad, the insurance office, the joint-stock company; our law, our primary assemblies, our commerce, the galvanic battery, the electric jar, the prism, and the chemist's retort; in which we seek now only an economic use. Is it not the selfish and even cruel aspect which belongs to our great mechanical works, to mills, railways, and machinery, the effect of the mercenary impulses which these works obey? When its errands are noble and adequate, a steamboat bridging the Atlantic between Old and New England and arriving at its ports with the punctuality of a planet, is a step of man into harmony with nature. The boat at St. Petersburg, which plies along the Lena by magnetism, needs little to make it sublime. When science is learned in love, and its powers are wielded by love, they will appear the supplements and continuations of the material creation."

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

DEXTER BRACKETT, M. Am. Soc. C. E.*

DIED AUGUST 26TH, 1915.

Dexter Brackett, the only son of Cephas Henry and Louisa Thwing (Pierce) Brackett, was born on November 30th, 1851, in Newton, Mass. During his childhood his parents removed to the adjoining town of Brighton, which subsequently became a part of the City of Boston. He was educated in the town schools and was graduated from the Brighton High School in 1868. In 1868-69 he took a six months' course at a business college.

As is the case with many young men, he had no well-defined plan as to his life work, but, in March, 1869, when he was 17 years old, an opportunity was presented for his employment in the office of the City Engineer of Boston, and as engineering appeared to him to be a desirable line of work, he accepted the position. The time proved to be a favorable one, as the City Engineer's office had been recently re-organized, and the great increase in the territory of the city, by the annexation of several towns, one after another, from 1868 to 1874, caused a large increase in the engineering work to be done, with corresponding opportunities for the promotion of those who had energy and ability. Only one of the five municipalities annexed was provided with a public water supply, and Mr. Brackett was soon assigned to the work incident to the extension of the pipes into these municipalities, and the establishment of a new high-service system of supply. The great fire in Boston, in November, 1872. also called urgent attention to the necessity for a more adequate system of main pipes in the city proper.

One of the most noteworthy events in the early part of Mr. Brackett's professional career, as well as in the careers of many of his associates, was the election, in the spring of 1873, of Joseph P. Davis, M. Am. Soc. C. E., as City Engineer of Boston. His personality and the high grade of his professional work moulded to a large extent the subsequent careers of those under him. The office served as a most valuable training school, both in engineering and in ethics, a fact which was fully recognized by Mr. Brackett during his life, as well as by many others among his associates.

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* Memoir prepared by Frederic P. Stearns, Past-President, Am. Soc. C. E.

Under Mr. Davis' direction, Mr. Brackett was early placed in charge of the distribution system of the water-works. In addition to the large amount of work done in extending and reinforcing the water-pipe system in these years, the question of the waste of water soon came to the front, on account of the increasing water consumption, and, as early as 1878, Mr. Brackett began a study of this problem for Mr. Davis. At that time, although the waste of water was recognized as a very important factor, it was not nearly as large as in subsequent years. Water meters were then much more expensive and less durable than at the present time, so that the remedial measures proposed were in the line of the restriction of waste through its detection by inspection, aided by the use of Deacon meters. Church stop-cocks and Bell water-phones were also tried as aids in this work.

During the next few years, Deacon meters were set in some districts of the city, and a very marked reduction in the consumption and waste of water in those districts resulted from their use. As the result of this and other work along these lines, Mr. Brackett took especial interest in matters relating to the consumption and waste of water, and became an authority on the subject. In addition to material prepared for the official city reports, he presented papers on the subject to the New England Water Works Association in 1886* and to the American Society of Civil Engineers in 1895.†

In 1895, he was asked by the State Board of Health of Massachusetts to advise as to the present and future consumption of water per inhabitant in the Metropolitan Water District, and made a valuable report‡ on this subject. In regard to this report, it may be stated that his predictions made in 1894, as to the probable water consumption per inhabitant 30 years later, now seem likely to be verified.

In 1904, he made a very complete report to the Metropolitan Water and Sewerage Board§ on the measurement, consumption, and waste of water supplied to the Metropolitan Water District, which was transmitted to the State Legislature as a basis for legislative action in regard to the restriction of waste. This will be referred to subsequently.

During Mr. Brackett's long connection with the Boston Water-Works, which continued until August 1st, 1895, he had experience with every phase of the distribution department of a water-works. His work included, not only engineering, but practical work as Superintendent in direct charge of the constructing and operating forces, from Febru-

* "Water Waste", *Journal,* New England Water Works Assoc., Vol. I, No. 2, p. 10 (December, 1886).

† "Consumption and Waste of Water", *Transactions,* Am. Soc. C. E., Vol. XXXIV, p. 185.

‡ Special Report, State Board of Health of Massachusetts, on a Metropolitan Water Supply, 1895, p. 157.

§ Annual Report, Metropolitan Water and Sewerage Board, January 1st, 1904, p. 301; also, *Journal,* New England Water Works Assoc., 1904, Vol. XVIII, p. 107.

ary, 1888, to June, 1891, and covered such features as the laying of pipes across navigable streams, the raising of large mains when full of water, the cleaning of tuberculated pipe, the testing of fire engines and fireboats, the construction of a high-pressure system of piping for fire protection only, the installation and testing of heavy pumping machinery, and the construction of distributing reservoirs.

Mr. Brackett often visited other cities to gain information as to improvements in water-works practice adopted by them, and to note the character of their works as compared with those of Boston. One of his notable studies of this kind was a comparison of the capacity of the distribution system of Boston with those of the cities of New York, Chicago, Philadelphia, Baltimore, St. Louis, Cincinnati, Cleveland, Pittsburgh, Washington, Brooklyn, and Detroit, made and reported on in 1892. It was his aim to keep the portion of the water-works under his charge up to the highest standard of practice, and in this he succeeded.

On August 1st, 1895, the engineering force of the Metropolitan Water-Works was organized, and Mr. Brackett, on account of his pre-eminent qualifications for the place, was selected as Engineer of the Distribution Department, in charge of the construction of the pipe lines, pumping stations, and reservoirs in the Metropolitan Water District. The essential parts of the works were planned and executed rapidly and well, and had advanced far enough by January 1st, 1898—the date fixed in the legislative act for the introduction of the Metropolitan water—to permit the water to be conveyed to all parts of the District on that day.

The pumping stations constructed and maintained for many years under the direction of Mr. Brackett are regarded by many who have visited them as models in respect to economy and efficiency.

On February 1st, 1907, Mr. Brackett became Chief Engineer of the Metropolitan Water-Works, and held this position until his death on August 26th, 1915. In this capacity, his duties related largely to the maintenance and operation of works already constructed, a kind of work which he enjoyed. He was very successful in selecting his assistants in charge of the various portions of the works, and, under his supervision, an extremely efficient and loyal force was developed. There were, however, during this time important additions to the works, among which may be mentioned the hydro-electric plant at the Wachusett Dam at Clinton, Mass., which has been very successful both mechanically and financially, and a similar plant at the Sudbury Dam, at Southborough, now under construction, which was designed under Mr. Brackett's direction.

One important reform took place on the Metropolitan Water-Works after Mr. Brackett was appointed Chief Engineer, namely, the introduction of water meters. This resulted largely from reports written

by him on the subject of consumption and waste of water, and from his persistent advocacy of the use of meters within the Metropolitan Water District as a means of reducing waste. He had the privilege of seeing his views adopted and the consumption of water thereby greatly decreased. Up to 1907, the consumption had been increasing so rapidly as to point out the necessity for an additional supply within the next few years, but the gradual introduction of meters reduced it so rapidly that no additional supply will be needed for many years; it is now less than two-thirds of what it would have been had the former rate of increase continued.

In addition to his regular engineering work, Mr. Brackett was called as an expert in connection with water supplies at Fall River, Stoughton, Taunton, New Bedford, Ashburnham, and Springfield, Mass.; Auburn, Me.; Syracuse, N. Y.; Atlantic City, N. J.; Louisville, Ky.; Springfield, Mo.; Harbor Springs, Mich., and other places, in several cases in connection with the testing of pumping engines or the valuation of the works.

He ceased to accept the many engagements offered him along these lines after 1895, at first because the regular work on which he was engaged demanded all his time, and later, because a diminishing vitality made it undesirable for him to accept the burden of work in addition to that required by his regular duties.

In all, Mr. Brackett served the City of Boston and the Metropolitan Water District for 46 years, most of the time in positions of much responsibility. Taking into account his entire trustworthiness, his loyalty to his work, and his special ability in the branches of water-works engineering in which he was engaged, it is difficult to over-estimate the value of the public service which he rendered during his long professional career.

He had a large circle of friends, both inside and outside of the Profession, who will miss him greatly. One of his associates in the New England Water Works Association, in 1904, in a paper entitled "Some Reminiscences",* refers to him thus:

"Dexter Brackett, who assumed the presidency at this time [1889], is an ex-president who is still young, very much alive, and a familiar figure to you all, and, what is still more to the purpose, he proposes to keep up his youthful ways and indomitable energy as long as he lives. He put a tremendous amount of labor into the work of his administration. There was no lack of inspiration while his hand was at the helm. He was a working president in every sense that the word implies. His deep interest in our affairs continues unabated, and the many helpful services which he is constantly rendering are always appreciated."

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* By R. C. P. Coggeshall, *Journal*, New England Water Works Assoc., Vol. XVIII, p. 313.

It is worthy of note that at the Convention of the New England Water Works Association, held a short time after his death, there were many members of the Association who thought Mr. Brackett's name should be perpetuated by some appropriate form of memorial, and suggested the raising of a fund, the annual income of which might be used as a prize for a meritorious paper, or otherwise. A resolution was adopted by the Convention providing for the appointment of a committee to consider the subject.

Mr. Brackett was President of the Boston Society of Civil Engineers in 1897-98; President of the New England Water Works Association, 1889-90; and a member of the American Water Works Association, and the Boston City Club.

Some of the more important papers contributed by him to the technical societies, in addition to those on the subject of the consumption and waste of water already referred to, related to the raising or otherwise moving of large water mains, the consumption of water at a large fire in Boston on November 28th, 1889, the tuberculation of water pipes, the freezing of fresh water in a pipe submerged in salt water, and a description of the Metropolitan Water Works presented to the American Water Works Association. He was an important member of the Committee of the New England Water Works Association which prepared standard specifications and drawings for cast-iron water pipe and special castings, which were adopted by the Association on September 10th, 1902.

On September 21st, 1875, Mr. Brackett was married to Miss Josephine Dame, who survives him. He also leaves one son.

Mr. Brackett was elected a Member of the American Society of Civil Engineers on June 6th, 1888. He also served as a Director of the Society from 1908 to 1910, inclusive.

FOSTER CROWELL, M. Am. Soc. C. E.*

Died March 29th, 1915.

Foster Crowell was born at Westchester, Pa., on October 13th, 1848. His ancestry traces back for more than 200 years to the earlier immigration to America. His great-grandfather was an early American sea captain, his grandfather, a schoolmaster in Philadelphia, and his father, the Rev. John Crowell, was an able and much beloved Presbyterian minister, long the pastor of the historic Brick Church, of East

* Memoir prepared by S. Whinery, M. Am. Soc. C. E.

Orange, N. J. His mother was Katharine Roney, of North of Ireland ancestry, whose father was an American soldier in the War of 1812.

Foster Crowell's school education began in his grandfather's school in Philadelphia, and was continued in a private school in Orange, N. J. In 1863, he entered the Pennsylvania Polytechnic College, from which he was graduated in 1867 with the degree of B. C. E. Three years later the degree of M. C. E. was conferred on him by the same institution. Before entering college, at the early age of 14 (1862), he enlisted in the Home Guard of New Jersey, and saw actual military service in the draft riots of that time.

Mr. Crowell began his professional career in 1867 as a Chainman on the engineering work for the improvement of Fairmount Park, Philadelphia. He was advanced to the position of Assistant Engineer in 1868, and was in charge of the construction work, with the title of Superintending Engineer, from 1870 to 1872.

In March, 1872, he was appointed one of the two civil engineers (the other being the late A. G. Menocal, M. Am. Soc. C. E.), with the United States Government expedition, under the auspices of the Navy Department, to make examinations and surveys for the Nicaragua Canal. In the capacity of Principal Assistant Engineer, he accompanied two expeditions to that then almost unknown tropical wilderness.

In the preparatory work for the Centennial Exposition, in Philadelphia (1874-75), Mr. Crowell was Principal Assistant Engineer on the Exposition Grounds and Engineer on the ironwork in the construction of Memorial Hall and Horticultural Hall. Resigning in June, 1875, he accepted a position as Manager and Engineer of the Philadelphia Branch of the Cornell Iron Works, having charge, in this connection, of the work done by that Company on the elevated railroad construction then in progress in New York City.

In 1878, he opened an office in New York City for general professional practice. In 1880 and 1881, he was Chief Engineer of the Elizabeth City and Norfolk Railroad during its location and construction. Entering the service of the Pennsylvania Railroad Company in 1881, as Engineer on Construction, he had charge, for about 7 years, of the many important improvements made by the Company in that period. Among these were the building of the Philadelphia, Germantown, and Chestnut Hill Railroad, portions of the Pennsylvania and Schuylkill Valley Railroad, the Bedford and Bridgeport Branch, the new line from Cincinnati, Ohio, to Richmond, Ind., besides reconstruction work on the main line, additional trackage, and miscellaneous work.

From 1888 to his death, Mr. Crowell was engaged in practice as a Consulting Engineer in New York City. Among a large number of professional engagements during that period, as disclosed by official reports left on file in his office, the following may be noted:

Reports on the engineering treatment of the water-front of Toronto, Ont., Canada; on the Ravine du Sud, Haiti, for the Republic of Haiti; on the Delaware, Lake Erie and Western Railroad; on the Boston and Maine Railroad; on the Genesee Valley Railroad; on terminal docks at Charleston, S. C., for the South Carolina and Georgia Railroad; on the Buffalo and Susquehanna Railroad; on the Harlem River Driveway; on the condition of various railroad properties and on the filling in of the Hackensack Meadows for a prominent New York banking firm; on a projected railroad from Sacramento to Lake Tahoe, Cal.; and (in conjunction with the late George S. Morison, Past-President, Am. Soc. C. E.) on additional track facilities through Bergen Hill, N. J., for the Erie Railroad. During this period he designed and built (1900-01) the notable all-steel Redridge Dam, at Houghton, Mich.

Mr. Crowell took an active part in that body of prominent engineers who worked up and prepared the monumental report to the Merchants Association of New York on a future water supply for that City, himself preparing the Special Report (1900) on a supplementary salt-water supply for fire protection. He was an active and useful member of the Committee appointed by the same Association which, in 1903, reported on passenger transportation in the City of New York.

In 1897, he was offered, but declined, the position of Chief Engineer of the Nicaragua Canal Commission (the Walker Commission), and, in 1902, he declined the position of Chief Engineer of the Department of Bridges of New York City.

In 1907, he accepted the position of Commissioner of Street Cleaning of the City of New York, but apparently did not find the work congenial and resigned in 1909. Early in 1912, he was appointed Consulting Engineer to the President of the Borough of Queens, City of New York, serving in that capacity until his death.

Mr. Crowell became a member of the Institution of Civil Engineers of Great Britain in May, 1893. He was also a member of the Philadelphia Engineers Club from 1883 to 1893, and was one of the organizers, and a member of the first Board of Directors, of the Cincinnati Engineers Club, while engaged on railroad work in that vicinity.

During all his life, Mr. Crowell took a keen and active interest in upholding and advancing the Engineering Profession, and contributed liberally to its literature.

To the American Society of Civil Engineers he contributed three important papers, and its *Transactions* bear further witness to his wide interest in varied branches of the Profession, his name appearing in the discussion of about fifty papers other than his own. His name also appears as the translator from the Spanish of three papers read at the International Engineering Congress of the Columbian Exposition, Chicago, 1893, and from the French of two papers read at the

International Engineering Congress, St. Louis, 1904. The *Transactions* of the Engineers Club of Philadelphia contain four papers by Mr. Crowell. Nor was his literary work confined to the professional societies; for the *Engineering Magazine,* he prepared the following articles: "Railroad Facilities of Suburban New York" (1896); "Modern Wharf Improvement and Harbor Facilities" (1896); "How Holland was Made" (1895); and for *Scribner's Magazine,* "A Little Journey to Haytii" (1891).

In 1904, he delivered the address to the graduating class of Rensselaer Polytechnic Institute.

During his busy and useful professional life, Mr. Crowell did not neglect the civic and other duties and responsibilities of the good citizen. In Flushing, N. Y., his home since 1890, and where he died on March 29th, 1915, he took an active interest, and was an influential factor, in local industrial, civic, charitable, and social affairs.

He was a member, and President, for four years, of the Flushing Association, the leading local organization in originating and promoting civic improvements, and served as a Trustee of the Flushing Hospital, and as a member of the local Board of Health, for the same number of years. He was also a Trustee of the Flushing Library Association.

Mr. Crowell was married to Anna McKinstry Whiting, daughter of John Nicholas Whiting, of Orange, N. J., on January 27th, 1881. Mrs. Crowell and two sons, the Rev. John Whiting Crowell and Francis Stirling Crowell, Assoc. M. Am. Soc. C. E., are living. Another son, Thomas Sutherland Crowell, is deceased.

For those who had the pleasure of a more or less intimate acquaintance with Mr. Crowell, it is unnecessary to speak of his personal and professional traits of character. For others, they may be briefly characterized: A man of unquestioned integrity and honor; of high scholarly and professional attainments; of courteous, but quiet and dignified bearing; a loyal friend; broad and liberal minded, but safely conservative; a man of positive convictions; a careful, clear thinker, his conclusions were formed with deliberation, stated with confidence, and defended with vigor; devoted and loyal to his profession, and able, painstaking, and thorough in his professional work.

He will be remembered as an exemplary man, a useful citizen, and an honor to the Profession.

Mr. Crowell was elected a Member of the American Society of Civil Engineers on December 1st, 1880. He was a member of the Board of Direction for two years (1893-95), and was also a member of the Society's Special Committee on Standard Sections of Railroad Rails, whose report, in 1893, marked an era in the design of railroad rails, and has had a controlling influence with both engineers and American railways ever since.

HENRY CLAY DERRICK, M. Am. Soc. C. E.*

Died May 9th, 1915.

Henry Clay Derrick was born on January 13th, 1832, at Washington, D. C., where his father was in the service of the Federal Government under which he held many important positions, among them being that of Chief Clerk of the State Department during the Buchanan Administration. He was a personal friend of Henry Clay, for whom his son, Henry Clay Derrick, was named.

Henry Clay Derrick was graduated from Bolmar's Academy, West Chester, Pa., and began his engineering career, in 1851, as Resident Engineer, under the late Captain Andrew Talcott, on the construction of the Richmond and Danville Railroad. In 1853, he was appointed Resident Engineer on the Pittsburgh, Maysville, and Cincinnati Railroad, where he remained until 1855.

From 1855 to 1858, he served as United States Deputy Surveyor for the Territories of Kansas and Nebraska. In 1858, he was appointed Assistant Engineer at the United States Arsenal at Harper's Ferry, Va., and took part in the stirring events of October, 1859, when John Brown captured and, for a few hours, held the Arsenal.

When the Civil War broke out in 1861, Mr. Derrick joined the Confederate Army and rendered active and conspicuous service as Captain of Engineers in the Army of Northern Virginia, until the close of the war, when he settled in Halifax County, Virginia, and resumed the practice of his profession. From 1868 to 1874 he was engaged as follows: 1868 to 1869, Division Engineer of the Norfolk and Great Western Railroad; 1869 to 1870, Resident Engineer on the Western North Carolina Railroad; 1870 to 1871, Principal Assistant Engineer on the Richmond, Fredericksburg, and Potomac Railroad; 1871 to 1872, Principal Assistant Engineer on the Piedmont and Potomac Railroad, and 1872 to 1874, Engineer in Charge on the Washington, Cincinnati, and St. Louis Railroad.

In 1875, Colonel Derrick accepted the appointment of Lieutenant-Colonel of Engineers on the General Staff of the Egyptian Army, and served with distinction for three years. During that time, he surveyed the military railroad from Massowah, on the Red Sea, to Bahr Reza, in Abyssinia. He also took part in the disastrous campaign against King John of Abyssinia, during which the Egyptian Army was almost destroyed as a fighting force, the remnant being saved by the exertions

* Memoir prepared by the Secretary from information furnished by H. G. Prout and G. H. Derrick, Members, Am. Soc. C. E.

of a small group of American officers, of whom Colonel Derrick was one. He was afterward decorated, for gallant service in the field, by the Khedive Ismail, with the Turkish Imperial Order of Medjidieh.

In 1878, Colonel Derrick returned to the United States and resumed his engineering practice, being engaged as follows: 1878 to 1879, Chief Engineer of the Danville and New River Railroad; 1880 to 1881, Resident Engineer on the Chesapeake and Ohio Railroad; 1882 to 1884, Engineer in Charge of the Richmond and Allegheny, and the Louisville, New Orleans and Texas Railroads; 1886 to 1887, Engineer in Charge of the Lynchburg and Durham Railroad; and 1887 to 1888, Principal Assistant Engineer on the Roanoke and Southern Railroad. In 1888, he retired from active work in his profession and returned to his home at Houston, Va., where he lived quietly until his death on May 9th, 1915.

Colonel Derrick's professional career during his thirty-seven years of railroad service was typical of the life of the civil engineer in the United States during that period when railroad building was most active. The greater part of the Profession was engaged on preliminary survey, location, and construction, and as soon as one railroad was nearing completion, they went to another. It was a hard but interesting life, useful to the country and the Nation, but it seldom led to fame or fortune for the individual. Colonel Derrick's retirement from active practice coincided with the discontinuance of active railroad construction in the United States.

In writing of Colonel Derrick's personality, those who knew him best speak of him with affection and respect, dwelling especially on his modesty and simplicity, his courage, his high-mindedness, and on the essential dignity of his character. He was intensely loyal to the South, where he preferred to remain in quiet seclusion rather than seek a wider environment where his personality and endowments would have gained for him the high professional position which he deserved. In social circles his keen wit and humor were always evident and delightful, and as a host, he was the soul of geniality, courtesy, and hospitality.

Colonel Derrick is survived by his widow, who was Miss Martha Cosby, and two daughters and three sons. Two of the sons, Guy H. and Clarence Derrick, and a grandson, John Russell Derrick, are members of this Society.

He was a Member of the National Geographical Society, and for many years a faithful officer and communicant of St. John's Protestant Episcopal Church, at Houston, Va.

Colonel Derrick was elected a Member of the American Society of Civil Engineers, on October 5th, 1887.

CHARLES FRANCIS, M. Am. Soc. C. E.*

DIED APRIL 29TH, 1914.

Charles Francis was born in Lowell, Mass., on August 10th, 1842. He was the son of the late James Bicheno Francis, Hon. M. Am. Soc. C. E., the eleventh President of this Society.

Charles Francis was reared in Lowell, and pursued his studies there until 1860, when he entered Harvard University, from which he was graduated with the Class of 1864. In 1861, while at Harvard, Mr. Francis enlisted as a soldier of the Union Army and served his country as a member of Company F of the 44th Massachusetts Regiment, receiving his honorable discharge after one year's service.

After Mr. Francis was graduated from Harvard, he determined to follow his father's profession, and as he wished to become familiar with machine-shop practice, he entered the Lowell Machine Shop. After serving his apprenticeship as a machinist, he spent several years in his father's office, and was associated with him on some of his important work.

Later, Mr. Francis went to Chicago where he remained until the great fire of 1871, at which time he removed to California. There his training as a hydraulic engineer was of great value to him, and for nine years he was associated with various mining enterprises in that State.

From 1879 until 1883, Mr. Francis was employed by the Mexican Central Railroad, with headquarters at Guadalajara, Mexico, working under the direction of the late A. M. Wellington, M. Am. Soc. C. E.

In 1883, Mr. Francis returned to Lowell where he was again associated in business with his father until 1889. In that year he accepted employment with the Federal Government as Engineer in charge of the construction of a dam at Rock Island Arsenal.

It was at this time that Mr. Francis' acquaintance with Davenport, Iowa, began, as he resided there while in charge of the Government work, and, at its completion, his attachment to that city was such that he decided to make it his permanent home. He immediately became prominent in the public affairs of the city, serving as Commissioner of Public Works from 1891 to 1892, and afterward practising as a Construction Engineer. He also served as a member of the Iowa State Board of Health for seven years.

Soon after he came to Davenport, Mr. Francis became interested in the possibilities of the hydro-electric development of the Mississippi River at that point, and was associated with the Davenport Water Power Company as Chief Engineer for twelve years. During this

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* Memoir prepared by William H. Kimball, M. Am. Soc. C. E.

time he made a careful study of the project, and his investigation of it showed the painstaking care that was characteristic of his work.

Mr. Francis was prominent in various organizations in Davenport, served on many local boards, and always gave his services generously. Everything he undertook was well done.

He was a member of the Masonic Fraternity, a Knight Templar, and a Noble of the Mystic Shrine.

As a member of the Protestant Episcopal Church, Mr. Francis served on the Vestry and, for some time before his death, as Junior Warden. He was also President of the Men's Club of the church, and gave generously of his time and means for its support.

Mr. Francis' travels, his broad general knowledge, his alert mind, and his interest in people made him a most companionable man. He was one of those men for whom the Profession of Engineering meant an abiding, life-long interest, not because of its pecuniary reward, but because he was proud to belong to it. He saw the various ways in which his engineering knowledge could serve the community, and he gave freely of this knowledge.

When the American Society of Civil Engineers visited the Panama Canal, Mr. Francis was a most enthusiastic member of the party, and on his return gave several illustrated talks describing the trip and the work.

In 1869, Mr. Francis was united in marriage to Miss S. C. Crosby, of Lowell, Mass., who, with an only daughter, Fanny C., survives him.

Mr. Francis was elected a Member of the American Society of Civil Engineers on May 4th, 1892.

WARREN AUSTIN GATES, Jun. Am. Soc. C. E.*

Died May 24th, 1914.

Warren Austin Gates was born at Coxsackie, N. Y., on March 7th, 1885. He received his early training at the Coxsackie High School, from which he was graduated with a University Scholarship. He was a graduate of the Massachusetts Institute of Technology, in the Class of 1907, and received the degree of Bachelor of Science.

After completing his college course, Mr. Gates worked for a few months with the Buffalo Expanded Metal Company on construction work in Buffalo, N. Y., and in December, 1907, went with the General Fireproofing Company, of Youngstown, Ohio, remaining in the Reinforced Concrete Designing Department of this Company until July,

* Memoir prepared by C. J. Paterson, Assoc. M. Am. Soc. C. E.

1908, when his health failed, compelling him to live in the Adirondacks for some time.

In October, 1909, Mr. Gates started to work again, with the firm of Layton Smith and Hawk, Architects, of Oklahoma City, Okla., as Engineer and Superintendent on reinforced concrete construction. He remained with this firm until his death on May 24th, 1914.

When he first went to Oklahoma he held the position of Assistant Engineer, but at the end of a year, he became Chief Engineer in charge of all engineering work in the office. This was a position of trust and responsibility, his work including among other structures, the Houston High School, Houston, Tex., and all preliminary work on the Oklahoma State Capitol, the construction of which was started at about the time of his death.

Mr. Gates was an engineer of exceptional promise, and one who at all times had before him the highest ideals of his Profession. He was a sincere friend and a man who had the complete confidence of all those who were brought in touch with him in his work.

He was married in October, 1911, to Miss Marion W. Sloane, of East Orange, N. J., and is survived by his widow and one daughter.

Mr. Gates was elected a Junior of the American Society of Civil Engineers on June 30th, 1911.

PAPERS IN THIS NUMBER

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"THE CHERRY STREET BRIDGE, TOLEDO, OHIO." Clement E. Chase. (To be presented Dec. 1st, 1915.)
"IDEALISM AND ART IN ENGINEERING": Presidential Address. Charles D. Marx.

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OF THE

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OF

CIVIL ENGINEERS

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(INSTITUTED 1852)

VOL. XLI—No. 9

NOVEMBER, 1915

Edited by the Secretary, under the direction of the Committee on Publications.

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The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

House of the Society—220 West Fifty-seventh Street, New York.

Telephone Number........................1446 Circle.
Cable Address........................"Ceas, New York."

* Vacancy in chairmanship caused by the death of Austin Lord Bowman.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

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MINUTES OF MEETINGS

OF THE SOCIETY

October 20th, 1915.—The meeting was called to order at 8.30 P. M.; Director George W. Fuller in the chair; Chas. Warren Hunt, Secretary; and present, also, 97 members and 21 guests.

A paper by F. zur Nedden, Esq., entitled "Induced Currents of Fluids", was presented by the author.

The Secretary read communications on the subject from Messrs. Clemens Herschel and Carl George de Laval, and the paper was discussed by John C. Trautwine, Jr., Assoc. Am. Soc. C. E., and the author.

The following motion was offered:

"That it is the sense of this meeting that the Society take whatever action is necessary to bring before the Bureau of Standards at Washington, and the Engineering Foundation, the desirability of co-

operation among American scientists in experimenting on the subject of viscosity and induced currents of fluids, in order to avoid the duplication of work."

On motion, duly seconded, the matter was referred to the Board of Direction.

The Secretary announced the following transfers by the Board of Direction on September 20th, 1915:

FROM ASSOCIATE MEMBER TO MEMBER

GEORGE HENRY BLISS, Boise, Idaho
FRANK LEMUEL CLAPP, Boston, Mass.
EARLE TALBOT, New York City
JACKSON FRANKLIN WITT, Dallas, Tex.

The Secretary announced the following deaths:

AXEL SAMUEL FREDERICK BERQUIST, of Brooklyn, N. Y., elected Member, June 6th, 1906; died October 6th, 1915.

AUGUSTUS JAY DU BOIS, of New Haven, Conn., elected Junior, July 7th, 1875; Member, October 5th, 1892; died October 19th, 1915.

EDWARD GRAY, of Cripple Creek, Colo., elected Associate Member, May 1st, 1907; Member, December 6th, 1910; died October 2d, 1915.

WILLIAM BYRD KING, of Fort Worth, Tex., elected Member, October 7th, 1896; died October 11th, 1915.

OLAF RIDLEY PIHL, of Pittsburgh, Pa., elected Member, October 2d, 1889; died October 14th, 1915.

OSMAN FRED COLE, of Crossett, Ark., elected Associate Member, September 2d, 1908; died September 27th, 1915.

THOMAS HOVENDEN, of Philadelphia, Pa., elected Associate Member, July 9th, 1912; died September 19th, 1915.

EDWARD WOOLSEY COIT, of Los Angeles, Cal., elected Fellow, September 20th, 1872; died September 25th, 1915.

Adjourned.

November 3d, 1915.—The meeting was called to order at 8.30 P. M.; Mansfield Merriman, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 95 members and 10 guests.

The minutes of the meetings of September 15th and October 6th, and of the Annual Convention, September 16th, 1915, were approved as printed in *Proceedings* for October, 1915.

A paper by Karl R. Kennison, Assoc. M. Am. Soc. C. E., entitled "The Hydraulic Jump, in Open-Channel Flow at High Velocity", was presented by the Secretary, who also read communications on the subject from Messrs. B. F. Groat, H. B. Muckleston, and Frederic P. Stearns.

The paper was discussed also by Messrs. R. D. Johnson, Mansfield Merriman, and H. F. Dunham.

The Secretary announced the following deaths:

Edward Macaulay Hartrick, of Galveston, Tex., elected Member, February 1st, 1899; died August, 1915.

Walter Cox Bowen, of New Brunswick, N. J., elected Junior, December 31st, 1913; died May 8th, 1915.

Adjourned.

ANNUAL CONVENTION AND INTERNATIONAL ENGINEERING CONGRESS, 1915

It was the intention to print in this number of *Proceedings* the Report in full of the Business Meeting of the Convention, together with an account of the Excursions and Entertainments, and also of the International Engineering Congress, 1915, but it has been impossible to do this because some of the necessary information is still lacking. These matters will be published in the December *Proceedings*.

On the following pages a list of the 246 papers to be published by the Congress is printed for the information of the membership.

INTERNATIONAL ENGINEERING CONGRESS, 1915

LIST OF PAPERS TO BE PUBLISHED, SHOWING THEIR ARRANGEMENT IN ELEVEN VOLUMES.

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Panama Canal

Introductory Paper

G. W. Goethals, Maj.-Gen., Corps of Engrs., U. S. A., M. Am. Soc. C. E., Canal Zone, Panama.

Outline of Canal Zone Geology

Donald F. MacDonald, Washington, D. C.

Climatology and Hydrology of the Panama Canal

F. D. Willson.

Sanitation in the Panama Canal Zone

Charles F. Mason, Lt.-Col., Medical Corps, U. S. A., Canal Zone, Panama.

Preliminary Municipal Engineering at Panama

Henry Welles Durham, M. Am. Soc. C. E., New York, N. Y.

Municipal Engineering and Domestic Water Supply in the Canal Zone

George M. Wells, M. Am. Soc. C. E., Canal Zone, Panama.

Dry Excavation (Panama Canal)

G. W. Goethals, Maj.-Gen., Corps of Engrs., U. S. A., M. Am. Soc. C. E., Canal Zone, Panama.

Dredging in the Panama Canal

W. G. Comber, M. Am. Soc. C. E., Canal Zone, Panama.

Construction of Gatun Locks, Dam, and Spillway

W. L. Sibert, Brig.-Gen., Corps of Engrs., U. S. A., M. Am. Soc. C. E., San Francisco, Cal.

Method of Construction of the Locks, Dams, and Regulating Works in the Pacific Division of the Panama Canal

S. B. Williamson, M. Am. Soc. C. E., Denver, Colo.

Lock Gates, Chain Fenders, and Lock Entrance Caissons

Henry Goldmark, M. Am. Soc. C. E., New York, N. Y.

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The General Design of the Locks, Dams, and Regulating Works of the Panama Canal

H. F. Hodges, Brig.-Gen., Corps of Engrs., U. S. A., M. Am. Soc. C. E., Fort Totten, N. Y.

Design of the Lock Walls and Valves of the Panama Canal

L. D. Cornish, M. Am. Soc. C. E., Cincinnati, Ohio.

The Design of the Spillways of the Panama Canal

E. C. Sherman, M. Am. Soc. C. E., Boston, Mass.

Emergency Dams above Locks of the Panama Canal

T. B. Mönniche, M. Am. Soc. C. E., Canal Zone, Panama.

Hydraulics of the Locking Operations of the Panama Canal

R. H. Whitehead, Assoc. M. Am. Soc. M. E., Canal Zone, Panama.

The Reconstruction of the Panama Railroad

Frederick Mears, Lieut., U. S. A., M. Am. Soc. C. E., Washington, D. C.

Permanent Shops, Pacific Terminals—Panama Canal

H. D. Hinman, Assoc. M. Am. Soc. C. E., and A. L. Bell, Assoc. M. Am. Soc. M. E., Canal Zone, Panama.

Electrical and Mechanical Installations of the Panama Canal

Edward Schildhauer, M. Am. Soc. C. E., Fel. Am. Inst. E. E., M. Am. Soc. M. E., New York, N. Y.

Terminal Works, Dry Docks, and Wharves of the Panama Canal

H. H. Rousseau, Civil Engr., U. S. N., Assoc. M. Am. Soc. C. E., Canal Zone, Panama.

Note:—The general fee for membership in the Congress ($5.00) covers the index volume and one additional volume. Other volumes may be purchased on a sliding scale of prices: one extra volume costing $3.50 in paper covers ($3.75 in cloth), two volumes, $6.75 in paper ($7.25 in cloth), etc.

Applications for membership in the Congress will be received until the latter part of December, 1915, and should be addressed to William A. Cattell, Secretary, International Engineering Congress, 1915, Foxcroft Bldg., San Francisco, Cal., who will gladly answer further inquiries.—(*Secretary.*)

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F. H. Cooke, Civil Engr., U. S. N., M. Am. Soc. C. E., Canal Zone, Panama.

Aids to Navigation of the Panama Canal

W. F. Beyer, Milwaukee, Wis.

The Working Force of the Panama Canal

R. E. Wood, Maj., U. S. A., Ret., Wilmington, Del.

Purchase of Supplies for the Panama Canal

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Natural Waterways in the United States

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Charles Davis Jameson, M. Am. Soc. C. E., Washington, D. C., and Peking, China.

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Utilization of Underground Waters

G. E. P. Smith, M. Am. Soc. C. E., Tucson, Ariz.

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M. Nethersole, Simla, India.

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Elwood Mead, M. Am. Soc. C. E., M. Inst. C. E., Melbourne, Victoria, Australia.

Irrigation in Spain; Distribution Systems, Methods, and Appliances

J. C. Stevens, M. Am. Soc. C. E., Portland, Ore.

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J. C. Stevens, M. Am. Soc. C. E., Portland, Ore.

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*Deceased since the preparation of the paper.

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E. C. Meier, Phœnixville, Pa.

Compressed Air in the Arts and Industries
W. L. Saunders, M. Am. Soc. C. E., Pres., Am. Inst. M. E., M. Am. Soc. M. E., New York, N. Y.

Safety Engineering
Frederic Remsen Hutton, Past-Pres. and Hon. Secy. Am. Soc. M. E., New York, N. Y.

Motor Vehicles; Passenger Type
Ethelbert Favary, M. Soc. Auto. Engrs., M. Aero. Soc., New York, N. Y.

Motor Vehicles; Utility Type
A. J. Slade, M. Am. Soc. M. E., M. Soc. Auto. Engrs., Assoc. M. Am. Inst. E. E., New York, N. Y.

Motor Tractors
F. S. Davis, M. Soc. Auto. Engrs., Chicago, Ill.

VOLUME VII

Electrical Engineering and Hydro-Electric Power Development

ELECTRICAL ENGINEERING

Economics of Electric Power Station Design
H. F. Parshall, M. Inst. C. E., London, England.

Industrial Effect of Low Cost of Electric Energy
L. A. Ferguson, Past-Pres., Am. Inst. E. E., Chicago, Ill.

The Effect of Hydro-electric Power Transmission upon Economic and Social Conditions, with Special Reference to the U. S. of America
Frank G. Baum, M. Am. Soc. C. E., M. Am. Soc. M. E., M. Am. Inst. E. E., San Francisco, Cal.

The Electric Motor as an Economic Factor in Industrial Life
David B. Rushmore, M. Am. Soc. C. E., M. Am. Inst. E. E., M. Am. Soc. M. E., M. Am. Inst. M. E., Schenectady, N. Y.

The Influence of the Electric Motor on Machine Tools
A. L. De Leeuw, M. Am. Soc. M. E., Elizabethport, N. J.

Electric Welding
C. B. Auel, M. Am. Soc. M. E., Pittsburgh, Pa.

The Application of Electricity to the Heating of Metals
F. L. Bishop, M. Am. Inst. E. E., Pittsburgh, Pa.

VOL. VII—(*Continued*)

The Mechanical Problem of the Electric Locomotive

G. M. Eaton, M. Am. Inst. E. E., East Pittsburgh, Pa.

Effects of Electrolysis on Engineering Structures

Albert F. Ganz, Fel. Am. Soc. M. E., Hoboken, N. J.

On the Production of High Permeability in Iron

Ernest Wilson, London, England.

Electric Illuminants

S. H. Blake, M. Am. Inst. E. E., M. Illum. Eng. Soc., Assoc. M. Am. Inst. E. E., Schenectady, N. Y.

HYDRAULIC AND HYDRO-ELECTRIC POWER DEVELOPMENT.

The Water Power of Sweden

Sven Lübeck, Pres., Svenska Teknologforeningen, Stockholm, Sweden.

Electric Power in Canadian Industry

Charles H. Mitchell, M. Am. Soc. C. E., M. Inst. C. E., M. Can. Soc. C. E., Toronto, Ont., Canada.

Developments in Modern Water Turbine Practice

H. Zoelly, M. Soc. Swiss Engrs. and Archts., Zürich, Switzerland.

Water Wheels of Pressure Type

Arnold Pfau, M. Am. Soc. M. E., Milwaukee, Wis.

Hydraulic Power Development and Use

J. D. Galloway, M. Am. Soc. C. E., San Francisco, Cal.

Water Wheels of Impulse Type

W. A. Doble, M. Am. Soc. C. E., M. Am. Soc. M. E., V. D. I., M. Int. Soc. Test. Mat., San Francisco, Cal.

Canadian Hydraulic Power Development

Charles H. Mitchell, M. Am. Soc. C. E., M. Inst. C. E., M. Can. Soc. C. E., Toronto, Ont., Canada.

VOLUME VIII

Mining Engineering

Economic and Social Influence of Mining

W. H. Shockley, F. R. G. S., M. Am. Inst. M. E., M. M. and M. Soc. of Am., M. Inst. M. and M. of London, Palo Alto, Cal.

Valuation of Metal Mines and Prospects

T. A. Rickard, A. R. S. M., M. Am. Inst. M. E., M. Inst. M. and M., San Francisco, Cal.

The Valuation of Oil Lands and Properties

M. E. Lombardi, M. Am. Inst. M. E., San Francisco, Cal.

VOL. VIII—(*Continued*)

Valuation of Anthracite Mines

R. V. Norris, M. Am. Soc. C. E., M. Am. Inst. M. E., M. Am. Soc. M. E., Wilkes-Barre, Pa.

Valuation of Coal Lands

Samuel A. Taylor, M. Am. Soc. C. E., M. Am. Inst. M. E., Pittsburgh, Pa.

Evaluating Coal Properties in Western Canada

R. W. Coulthard, Calgary, Alberta, Canada.

Status of Coal Mines in France

E. Gruner, Paris, France.

Workmen's Compensation and Mine Safety

H. M. Wilson, M. Am. Soc. C. E., M. Am. Inst. M. E., Pittsburgh, Pa.

Functions and Work of Exploration and Development Companies

H. W. Turner, M. Am. Inst. M. E., San Francisco, Cal.

The Financing of Mines in the United States

Lucius W. Mayer, New York, N. Y.

European Mining Finance

J. L. Gallard, London, England.

Organization and Staff of Mining Companies

W. H. Shockley, F. R. G. S., M. Am. Inst. M. E., M. M. and M. Soc. of Am., M. Inst. M. and M. of London, Palo Alto, Cal.; and R. E. Cranston, M. Am. Inst. M. E., M. Am. Soc. M. E., San Francisco, Cal.

Relation of Governments to Mining

Horace V. Winchell, M. Am. Inst. M. E., Minneapolis, Minn.

Mine Inspection

J. W. Paul, Pittsburgh, Pa.

VOLUME IX

Metallurgy

Symposium on Iron and Steel

Edited by Henry M. Howe, M. Am. Inst. M. E., Bedford Hills, N. Y.

Iron and Steel Castings

John Howe Hall, M. Am. Inst. M. E., M. Am. Soc. Test. Mat., High Bridge, N. J.

The Duplex Process of Steel Manufacture

F. F. Lines, M. Am. Inst. M. E., Sparrows Point, Md.

Steel Making in the Electric Furnace

James H. Gray, M. Am. Inst. M. E., New York, N. Y.

Methods of Preventing Piping in Steel Ingots

Emil Gathmann, M. Am. Inst. M. E., Baltimore, Md.

VOL. IX—(*Continued*)

Steel Alloys
George L. Norris, Pittsburgh, Pa.

Case Hardening of Steel
F. Giolitti, Turin, Italy.

Metallography and the Hardening of Steel
Albert Sauveur, M. Am. Inst. M. E., Cambridge, Mass.

Symposium on Copper

Edited by E. P. Mathewson, M. Am. Inst. M. E., Anaconda, Mont.

Progress in Copper Metallurgy
Thomas T. Read, M. Am. Inst. M. E., San Francisco, Cal.

Metallurgy of Copper in Japan
R. Kondo, M. Am. Inst. M. E., Tokyo, Japan.

Copper Metallurgy of the Southwest
James Douglas, Past-Pres. and Hon. M. Am. Inst. M. E., New York, N. Y.

Advances made in the Metallurgy of Copper, Globe District, Arizona
L. O. Howard, M. Am. Inst. M. E., Globe, Ariz.

Advances in Copper Smelting
Frederick Laist, M. Am. Inst. M. E., Anaconda, Mont.

Improvements in Design and Construction of Modern Copper Plants
Charles H. Repath, M. Am. Soc. M. E., Los Angeles, Cal.

Reduction Works—Copper Queen Consolidated Mining Company, Douglas, Arizona
Forest Rutherford, M. Am. Inst. M. E., Douglas, Ariz.

The Development of Electrolytic Copper Refining
Lawrence Addicks, M. Am. Soc. M. E., M. Am. Inst. M. E., Assoc. Am. Inst. E. E., Pres. Am. Electrochem. Soc., New York, N. Y.

Electrolytic Refined Copper
A. C. Clark, M. Am. Inst. M. E., Perth Amboy, N. J.

Leaching Copper Ores
W. L. Austin, M. Am. Inst. M. E., Riverside, Cal.

Physical Properties of Copper
Carle R. Hayward, M. Am. Inst. M. E., Boston, Mass.

Boronized Cast Copper
E. Weintraub, West Lynn, Mass.

The Metallography of Copper
William Campbell, M. Am. Inst. M. E., New York, N. Y.

VOL. IX—(*Continued*)

Symposium on Gold and Silver

Edited by C. W. Merrill, M. Am. Inst. M. E., San Francisco, Cal.

Coarse Crushing Plant; 1 000 tons Capacity
G. O. Bradley, San Francisco, Cal.

Crushing and Grinding
L. D. Mills, M. Am. Inst. M. E., and M. H. Kuryla, M. Am. Inst. M. E., San Francisco, Cal.

Solution of Gold and Silver
M. H. Kuryla, M. Am. Inst. M. E., San Francisco, Cal.

Filtration or Separation of Metal Bearing Solution from Slime Residue
L. D. Mills, M. Am. Inst. M. E., San Francisco, Cal.

Precipitation
G. H. Clevenger, M. Am. Inst. M. E., Leland Stanford Jr. University, Cal.

The Smelting and Refining of Lead
H. O. Hofman, M. Am. Inst. M. E., Boston, Mass.

Symposium on the Metallurgy of Zinc

Edited by Walter Renton Ingalls, M. Am. Inst. M. E., New York, N. Y.

Some Main Points in the Economics of the Metallurgy of Zinc
Walter Renton Ingalls, M. Am. Inst. M. E., New York, N. Y.

The Development of Zinc Smelting in the United States
George C. Stone, New York, N. Y.

Ore Dressing
Robert H. Richards, M. Am. Inst. M. E., Boston, Mass.

Electro-Metallurgy
E. F. Roeber, M. Am. Inst. M. E., New York, N. Y.

Metallography and Technology of Non-Ferrous Alloys
William Campbell, M. Am. Inst. M. E., New York, N. Y.

Symposium on the Utilization of Fuels in Metallurgy

Edited by C. H. Fulton, M. Am. Inst. M. E., Cleveland, Ohio.

Pulverized Coal in Reverberatory Furnaces
D. H. Browne, M. Am. Inst. M. E. New York, N. Y.

VOL. XI—(*Continued*)

Principles and Theories of Aerodynamics

J. C. Hunsaker, Lieut., Naval Constructor, U. S. N., Boston, Mass.

Refrigeration

Refrigeration

J. F. Nickerson, M. Am. Assoc. of Refrigeration, Chicago, Ill.

Refrigeration

Thor Andersson, Stockholm, Sweden.

Refrigeration in France

L. Marchis, Paris, France.

Agricultural Engineering

Agriculture and the Engineer

J. B. Davidson, M. Am. Soc. A. E., M. Am. Soc. M. E., Ames, Iowa.

Some Observations on the Extent and Value of Farm Power Equipment

Philip S. Rose, M. Am. Soc. Agri. Engrs., Madison, Wis.

Engineering Education

Some Considerations Regarding Engineering Education in America

George F. Swain, Past-Pres. Am. Soc. C. E., M. Am. Soc. M. E., Cambridge, Mass.

Technical Education for the Professions of Applied Science

Ira N. Hollis, M. Am. Soc. M. E., Worcester, Mass.

VOL. XI—(*Continued*)

Heating and Ventilation

Introductory Paper

R. C. Carpenter, M. Am. Soc. M. E., Ithaca, N. Y.

Recent Developments in Heating and Ventilation Art

D. D. Kimball, Pres. Am. Soc. Heat. and Vent. Engrs., New York, N. Y.

Vacuum, Vacuo-Vapor and Atmospheric Heating Systems

J. D. Hoffman, M. Am. Soc. M. E., Lincoln, Nebr.

Scientific Management

Developments and Progress in "Scientific Management" During Recent Years

E. P. Lesley, M. Am. Soc. M. E., Leland Stanford Jr. University, Cal.

Industrial Management

A. Hamilton Church, New York, N. Y.

Motion Study and Time Study Instruments of Precision

Frank B. Gilbreth, M. Am. Soc. M. E., and Lillian Moller Gilbreth, Providence, R. I.

Symposium on the Status of Engineering in Chile

Contributed through G. Lira, Santiago, Chile.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

December 1st, 1915.—8.30 P. M.—This will be a regular business meeting. Two papers will be presented for discussion, as follows: "The Automatic Volumeter", by E. G. Hopson, M. Am. Soc. C. E.; and "The Cherry Street Bridge, Toledo, Ohio", by Clement E. Chase, Jun. Am. Soc. C. E.

These papers were printed in *Proceedings* for October, 1915.

December 15th, 1915.—8.30 P. M.—At this meeting a paper by Benjamin F. Groat, M. Am. Soc. C. E., entitled "Chemi-Hydrometry and Its Application to the Precise Testing of Hydro-Electric Generators", will be presented for discussion.

This paper is printed in this number of *Proceedings.*

January 5th, 1916.—8.30 P. M.—A regular business meeting will be held, and a paper by William P. Creager, M. Am. Soc. C. E., entitled "The Economical Top Width of Non-Overflow Dams", will be presented for discussion.

This paper is printed in this number of *Proceedings.*

ANNUAL MEETING

The Sixty-third Annual Meeting will be held at the Society House, on Wednesday and Thursday, January 19th and 20th, 1916. The Business Meeting will be called to order at 10 o'clock on Wednesday morning. The Annual Reports will be presented, officers for the ensuing year elected, members of the Nominating Committee appointed, Reports of Special Committees presented for discussion, and other business transacted.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be per-

formed quite as well, and much more quickly, by persons familiar with the library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In making a search it sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is small. This method is particularly useful when there are drawings or figures in the text, which would be very expensive to reproduce by hand.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings.*

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings.*

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at

6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M., every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

(Abstract of Minutes of Meeting)

August 20th, 1915.—The meeting was called to order; Vice-President Couchot in the chair; E. T. Thurston, Jr., Secretary; and present, also, 66 members and guests.

On motion, duly seconded, the meeting adopted the recommendation of the Board of Directors that the Association participate in a material way in the entertainment of visiting members of the Society by assuming the responsibility of the reception, dinner, and dance to be given at the Old Faithful Inn, on September 16th, 1915.

Mr. C. E. Grunsky, Chairman of the Committee of Arrangements for the Annual Convention, spoke of alternate excursions to the Del Monte trip provided by the Engineering Congress.

Messrs. C. H. Snyder, Frederick R. Muhs, and William H. Popert, were appointed by the Chair as the Entertainment Committee for the next meeting.

Mr. E. F. Kriegsman addressed the meeting on the advantages to the engineer of proficiency in public speaking, and, on motion, duly seconded, a suggestion that the Association make provision for instructing its members in public speaking, was referred to the Board of Directors.

A paper entitled "Types of Country Roads", by Mr. M. O. Eldridge, was presented by the author, who illustrated his remarks with stereopticon views.

Adjourned.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, L. R. Hinman, 1400 West Colfax Ave., Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, at 12.30 P. M., at Clarke's Restaurant, 1632 Champa Street.

Visiting members are urged to attend the meetings and luncheons.

Atlanta Association

The Atlanta Association of Members of the American Society of Civil Engineers was organized on March 14th, 1912. The Association holds its meetings at the University Club, Atlanta, Ga.

At the meeting of the Association on January 9th, 1915, the following officers were elected for the ensuing year: President, Park A. Dallis; First Vice-President, B. M. Hall; Second Vice-President, P. H. Norcross; Secretary-Treasurer, T. B. Branch.

Baltimore Association

On May 6th, 1914, the Baltimore Association of Members of the American Society of Civil Engineers was organized, a Constitution adopted, and the following officers were elected: J. E. Greiner, President; Francis Lee Stuart, First Vice-President; L. H. Beach, Second Vice-President; Harry D. Williar, Jr., Secretary-Treasurer; and Messrs. H. D. Bush, B. T. Fendall, B. P. Harrison, Calvin W. Hendrick, Oscar F. Lackey, M. A. Long, and A. A. Thompson, Directors.

At its meeting of September 2d, 1914, the Board of Direction considered and approved the proposed Constitution of the Baltimore Association of Members of the American Society of Civil Engineers.

Cleveland Association

The proposed Constitution of the Cleveland Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on January 6th, 1915.

The following officers have been elected: President, Willard Beahan; Vice-President, Robert Hoffmann; Secretary-Treasurer, George H. Tinker.

Louisiana Association

At the meeting of the Louisiana Association of Members of the American Society of Civil Engineers (New Orleans, La.), on April 14th, 1915, the following officers were elected for the ensuing year: J. F. Coleman, President; W. B. Gregory and A. M. Shaw, Vice-Presidents; Ole K. Olsen, Treasurer; and E. H. Coleman, Secretary.

Northwestern Association

The proposed Constitution of the Northwestern Association of Members of the American Society of Civil Engineers (St. Paul and Minneapolis, Minn.) was considered and approved by the Board of Direction of the Society on November 4th, 1914. F. W. Cappelen is President and R. D. Thomas, Secretary.

Philadelphia Association

The meetings of the Philadelphia Association of Members of the American Society of Civil Engineers are held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

The officers of the Association are as follows: President, Edward B. Temple; Vice-Presidents, Edgar Marburg and John Sterling Deans; Directors, J. W. Ledoux, H. S. Smith, Henry H. Quimby, and George A. Zinn; Past-Presidents, George S. Webster and Richard L. Humphrey; Treasurer, S. M. Swaab; and Secretary, W. L. Stevenson.

(Abstract of Minutes of Meeting)

October 4th, 1915.—The Annual Meeting was called to order at the Engineers' Club; President Richard L. Humphrey in the chair; W. L. Stevenson, Secretary; and present, also, 50 members.

The Annual Reports of the Secretary and Treasurer were accepted and approved.

A memoir of the late William Hunter, M. Am. Soc. C. E., prepared by a Committee consisting of Messrs. George S. Webster (Chairman), Edward B. Temple, and Samuel T. Wagner, was read and approved.

Dr. John A. Brashear, President of the American Society of Mechanical Engineers, delivered an illustrated address on "Contributions of Photography to Our Knowledge of the Stellar Universe".

A vote of thanks was extended to Dr. Brashear.

The report of the Tellers was presented, showing the unanimous election of the following officers for the ensuing year: President, Edward B. Temple; Vice-President, John Sterling Deans; Directors, Henry H. Quimby and George A. Zinn; and Treasurer, S. M. Swaab.

Mr. Temple was installed as President.

Adjourned.

Portland, Ore., Association

At the Annual Meeting of the Association on September 28th, 1915, the following officers were elected for the ensuing year: President, J. P. Newell; First Vice-President, John T. Whistler; Second Vice-President, E. B. Thomson; Treasurer, Russell Chase; and Secretary, J. A. Currey.

St. Louis Association

The proposed Constitution of the St. Louis Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on October 7th, 1914.

The following officers have been elected: President, J. A. Ockerson; First Vice-President, Edward E. Wall; Second Vice-President, F. J. Jonah; Secretary-Treasurer, Gurdon G. Black. The meetings of the Association are held at the Engineers' Club Auditorium.

San Diego Association

The San Diego Association of Members of the American Society of Civil Engineers was organized on February 5th, 1915, and officers have been elected, as follows: President, George Butler; Vice-President, Willis J. Dean; and Secretary-Treasurer, J. R. Comly.

At its meeting of September 20th, 1915, the Board of Direction considered and approved the proposed Constitution of the San Diego Association of Members of the American Society of Civil Engineers.

Seattle Association

The Seattle Association of Members of the American Society of Civil Engineers was organized on June 30th, 1913. At its meeting of January 25th, 1915, the following officers were elected for the ensuing year: President, R. H. Ober; Vice-President, A. S. Downey; and Secretary-Treasurer, Carl H. Reeves.

(Abstract of Minutes of Meetings)

September 27th, 1915.—The meeting was called to order at 12.15 P. M.; President R. H. Ober in the chair; Carl H. Reeves, Secretary; and present, also, 33 members and guests.

The minutes of the meeting of August 30th, 1915, were read and approved.

On motion, duly seconded, President Ober appointed Messrs. S. H. Hedges, Ernest B. Hussey, and A. O. Powell, a committee to suggest the name or names of members for the Nominating Committee of the Society from this District.

Messrs. John R. Freeman, Charles F. Loweth, and E. E. Haskell addressed the meeting on the work of the United Engineering Societies and on the need of closer affiliation of all the National engineering societies.

Addresses were also made by Col. J. B. Cavanaugh and Prof. Milnor Roberts, the latter giving a brief outline of the work of the International Engineering Congress, at San Francisco, Cal.

Adjourned.

October 25th, 1915.—The meeting was called to order at 12.15 P. M.; President Ober in the chair; Carl H. Reeves, Secretary; and present, also, 17 members and guests.

The minutes of the meeting of September 27th, 1915, were read and approved.

In accordance with the Resolution passed at the meeting of August 30th, 1915, the President appointed Messrs. Robert Howes and John L. Hall, as members of the Conference Committee, to act for the Association in connection with the signing of the Articles of Agreement for the affiliation of the local engineering societies.

A letter from the Secretary of the Puget Sound Section of the American Chemical Society announcing its adoption, with minor changes, of the Articles of Agreement was read, and Secretary Reeves announced that similar action had been taken by the Local Section of the American Institute of Electrical Engineers.

W. Edward Wilson, ex-Chief Engineer of the International Waterways Commission, addressed the meeting briefly on the work of that Commission.

Mr. T. H. Carver was admitted to membership in the Association.

Mr. S. H. Hedges suggested that other local associations be notified of the Annual Meeting and banquet of the Association.

Messrs. A. M. Sargent and A. H. Dimock addressed the meeting briefly on the Lake Washington Canal Locks and on the status of the City bridges over the Lake Washington Canal.

Mr. Hedges reported progress on the matter of re-districting the membership of the Society.

Adjourned.

Southern California Association

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bimonthly meetings, with banquet, on the second Wednesday of Feb-

ruary, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary of the Association, W. K. Barnard, 514 Central Building, Los Angeles, Cal.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

Spokane Association

The proposed Constitution of the Spokane Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on March 4th, 1914. Ulysses B. Hough is President.

Texas Association

The proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on December 31st, 1913. The headquarters of the Association is Dallas, Tex. John B. Hawley is President.

MINUTES OF MEETINGS OF SPECIAL COMMITTEES TO REPORT UPON ENGINEERING SUBJECTS

Special Committee on Floods and Flood Prevention

August 30th, 31st, and September 1st, 1915.—The meetings were held in St. Louis, Mo., at the office of the President of the Mississippi River Commission. Present, C. McD. Townsend (Chairman), Daniel W. Mead, John A. Ockerson, and F. L. Sellew.

Letters from other members of the Committee were read and considered, and a report was prepared to be submitted to the next regular meeting of the Society.

Special Committee on Valuation of Public Utilities

October 11th, 12th, 13th, and 14th, 1915.—Ten meetings were held at the Society House. Present, F. P. Stearns (Chairman), H. E. Riggs, W. G. Raymond, C. S. Churchill, and J. P. Snow.

Eight of the ten meetings were devoted largely to a discussion of the subject of Depreciation and the other two to Cost of Reproduction.

It was decided that the next meeting of the Committee should be held during the latter part of December, 1915.

Special Committee on Materials for Road Construction

October 23d, 1915.—The meeting was held at the House of the Society. Present, W. W. Crosby (Chairman), Nelson P. Lewis, Charles J. Tilden, and A. H. Blanchard (Secretary).

The minutes of the meeting of August 13th, 1915, were read and approved.

The tentative draft of the 1915 Report, containing conclusions proposed by the sub-committees on the several non-bituminous road materials, was presented by the Chairman.

The sections of the Report covering general conclusions pertaining to non-bituminous road materials and specific conclusions relative to broken stone and slag roadways, gravel roadways, cement-concrete pavements, and brick and slag block pavements, were tentatively adopted.

It was decided that the next meeting of the Committee should be held at the House of the Society on November 4th, 1915.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten=Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston. Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 176 Mansfield Street, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Detroit Engineering Society, 46 Grand River Avenue, West, Detroit, Mich.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, 6 West Eager Street, Baltimore, Md.

Engineers' Club of Kansas City, E. B. Murray, Secretary, 920 Walnut Street, Kansas City, Mo.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Club of Trenton, Trent Theatre Building, 12 North Warren Street, Trenton, N. J.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, The Minories, Tower Hill, London, E., England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

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Montana Society of Engineers, Butte, Mont.

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Pacific Northwest Society of Engineers, 312 Central Building, Seattle, Wash.

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EXAMPLES IN ALTERNATING-CURRENTS, VOL. I.

For Students and Engineers. By F. E. Austin. Leather, 7¾ x 5 in., illus., 223 pp. Hanover, N. H., The Author, 1915. $2.40.

One of the objects of this book, as stated in the preface, is to assist college students, as well as those pursuing correspondence courses, in Electrical Engineering, to apply fundamental principles in engineering practice by the solution of engineering problems. It is also intended, it is said, to supply the guidance needed by such students in their work and to lessen the labor of the teacher. The subject-matter is composed of definitions and examples as applied to the subject of alternating currents, together with problems and their solution. The author calls attention to various tabulated values contained herein, which, he states, save time in solving problems in class-work and in practice. Although the book is intended for students, the results obtained in the solution of many of the problems can be applied, it is said, to considerable advantage by mechanical, civil, or electrical engineers in ordinary practice. A partial list of Contents is: Notation; The Greek Alphabet; The Pythagorean Theorem; Trigonometrical Functions of the Sum and Difference of Two Angles; Tabulation of Trigonometrical Functions; Relations of Trigonometrical Functions of Double and Half Angles; Expressions Involving the Powers of Trigonometrical Functions; Relations of Trigonometrical Functions of Three or More Angles; Some Trigonometrical Relations; Resultant of Two Forces; Solution of Triangles; Rules for Determining the Elementary Functions Involving a Single Variable; Integration; Definitions; Frequency; Production of Electro-Motive Force; Average Value of Sine-Curve; Rate of Change of Sine-Curve Alternating-Quantities; How to Plot the "Curve of Squares" for a Circle; Methods of Finding the Areas of Curves; Effective or R. M. S. Values of Non-Sine Pressures; Instantaneous Values of Non-Sine Pressures and Currents; Resultant of Four Pressures; Addition of Sine-Pressures; Non-Sine Alternating Curves; etc., etc.

HOW TO MAKE LOW-PRESSURE TRANSFORMERS.

By F. E. Austin. Second Edition with Additions. Cloth, 7¼ x 4¾ in., illus., 17 pp. Hanover, N. H., The Author, 1915. 40 cents.

The Introduction states that, in answer to many inquiries regarding the design, construction, and operation of small transformers for experimental purposes, the author has given, in this book, working directions and data for making such transformers, from 100 to 400 watts, at small cost and without the use of expensive tools and machinery. In order to illustrate his theories clearly, he has described briefly and concisely the building of a "step-down" transformer, to reduce the pressure from 110 volts to about 8 volts as a minimum, which may be used in operating low-pressure tungsten lamps, motors, electric bells, sparking devices for gasolene engines, small arc-lights, etc. In this, the second, edition, much additional matter has been included, it is said, in answer to a number of questions pertaining to fundamental principles received by the author, from those interested in transformer construction, after the issue of the first edition.

DIRECTIONS FOR DESIGNING, MAKING, AND OPERATING HIGH-PRESSURE TRANSFORMERS.

By F. E. Austin. Cloth, 8 x 5 in., illus., 46 pp. Hanover, N. H., The Author, 1914. 65 cents.

This book is a companion volume to the author's "How to Make a Transformer for Low-Pressures", and has been written, it is stated, for the engineering student and for all those who build or operate high-pressure transformers. After a brief discussion of the types of transformers in use, which includes data on losses in a transformer, power factors, etc., the author outlines briefly the design and construction of a 20 000-volt transformer and includes data which may be applied in building high-pressure transformers for various experimental uses, such as wireless telegraphy, for the production of "ozone", vacuum tube lighting, etc. In this description the author has given all the necessary calculations and materials, as well as the approximate cost of the materials used. The mathematical treatment is simple and the subject-matter is fully illustrated. It is hoped that the book will serve as an aid to the student in grasping the general principles of transformer design and operation.

* Unless otherwise specified, books in this list have been donated by the publishers.

RIVINGTON'S NOTES ON BUILDING CONSTRUCTION:

A Book of Reference for Architects and Builders and a Text-Book for Students, Parts I-II. Edited by W. Noble Twelvetrees. New Edition, Entirely Rewritten. Cloth, 9 x 6 in., illus., 2 vol. London, New York, Longmans, Green and Co., 1915.

The preface states that the first edition of this work was published in 1875, and was intended primarily to assist students preparing for the examinations in Building Construction held under the direction of the Board of Education, South Kensington, London, England. In order, at this time, to bring the subject-matter up to date and to maintain its reputation as a standard text-book for students on the design, construction, and equipment of buildings and as a reference book for architects and builders, the book, it is stated, has been entirely re-written by well-known architects and others having special knowledge of the subjects, many new chapters having been added, together with numerous illustrations. The subject-matter is divided into two parts, Part I dealing, it is said, with matters preliminary to building operations and with various forms of construction by which buildings of different types can be constructed from foundation to roof level, and Part II with classes of work which relate to the finishing of buildings for occupation and with various kinds of sanitary and engineering equipment. Part II, it is stated, may also be found useful as an independent manual for plumbers and sanitary engineers. The Appendix contains selected Examination Questions from papers set by the Royal Institute of British Architects, the Board of Education, the City and Guilds of London Institute, etc., etc. There is also a List of Contributors whose names indicate, it is said, the authoritative sources of the work. The Contents are: Part I: Building Regulations; Sites and Foundations; Timbering Excavations, Shoring and Underpinning; Scaffolding; Centers and Moulds; Brickwork; Masonry; Walls, Piers, and Retaining Walls; Arches, Vaulting, and Domes; Chimneys, and Setting for Stoves, Ranges, and Boilers; Damp and Sound-Resisting Construction; Iron and Steel Work; Steel Skeleton Buildings; Reinforced Concrete, etc.; Fire-Resisting Construction; Carpentry; Partitions; Appendix; Index. Part II: Roofs and Roofing; Timber Roof; Steel Roofs; Roof Coverings; Structural Plumbing; Joinery; Windows and Glazing; Stairs and Staircases; Plastering; Painting and Decorating; Drainage and Sewage Disposal; Water Supply, Plumbing, and Sanitation; Sanitary Fittings; Heating, Ventilation, and Hot-Water Supply; Gas and Electric Lighting; Gas-Fitting and Electric-Light Installation; Electric Bells, Telephones, and Lightning Conductors; Fire Equipment; Appendix; Index.

MECHANICAL DRAWING FOR SCHOOLS AND UNIVERSITIES.

By James D. Phillips and Herbert D. Orth. Cloth, 9 x 6 in., illus., 283 pp. Chicago, New York, Scott, Foresman and Company, 1915.

Drawings are stated by the authors to be to-day one of the first steps in the production of practically all machines and structures, and, in this book, it has been their aim to arrange a course in drawing which will give the student some idea of the best commercial drafting practice in that subject. The course, as presented in this book, is the result of years of experience and test in teaching the subject and is stated to be a course in working drawings supplemented by lectures and problems for the college student without previous training in the subject. The material is presented, it is stated, in the order in which it would occur in a commercial drafting-room, and is arranged to distribute the theory and the use of the instruments so that the student will comprehend both. Attention is called to the subject of Perspective Sketching as treated in the first chapter. Each problem included has been chosen, it is said, to illustrate principles of representation, dimensions, etc., and with each orthographic problem a type problem is given, which consists of given data and the solution of a problem similar to that assigned to the student. Chapter X, Instructor's Guide, is intended, it is stated, to aid the instructor in securing the viewpoint of the author and to reinforce his individual method. The Chapter Headings are: Perspective Sketching; Orthographic Sketching; Pencil Mechanical Drawing; Tracing and Blueprinting; Instruments and Materials; Conventions; Lettering; Advanced Drawing; Auxiliary Views, Isometric and Cabinet Drawing, Tables, etc.; Instructor's Guide; Outline of Course in Mechanical Drawing; Index.

PRACTICAL SURVEYING:

For Surveyors' Assistants, Vocational, and High Schools. By Ernest McCullough, M. Am. Soc. C. E. Cloth, 7½ x 5 in., illus., 9 + 401 pp. New York, D. Van Nostrand Company, 1915. $2.00.

This book, the author states, is a serious attempt on his part to meet the needs of students in surveying, whose mathematical preparation does not extend beyond the arithmetic taught in grade schools, and it is intended, therefore, as a textbook for

high schools, vocational schools, and evening classes, and as an aid to self-tutored men who wish to become surveyors. The subject, it is stated, has been presented in a logical manner, the various methods of surveying being described, in popular style, together with the instruments used and their care, and, at the end of each chapter, are given problems relating to the matter discussed in that chapter. The mathematical instruction is given step by step as needed, it is stated, and there is a chapter on trigonometry which, it is said, contains the minimum knowledge of that subject which a surveyor should possess. The Appendix on the essentials of algebra should serve, it is stated, as a useful introduction to the study of algebra. The methods of land surveying have been emphasized, but sufficient engineering surveying has been included, it is said, to help local surveyors over hard places. There is a chapter on surveying law and practice which, the author states, should be of much use to the student and about which he seldom knows anything until he begins practical work. The names and prices of books dealing more fully with the subject discussed will be found in many places in the text. The Contents are: Introductory; Chain Surveying; Leveling; Compass Surveying; Trigonometry; Transit Surveying; Surveying Law and Practice; Engineering Surveying; Appendix A: The Essentials of Algebra.

THOMAS' REGISTER OF AMERICAN MANUFACTURERS

And First Hands in All Lines, October, 1915. Seventh Edition. Cloth, 12½ x 10¼ in., illus., 3 100 pp. New York, Thomas Publishing Company, 1915. $15.00.

In a secondary title, it is stated that this work is the largest classified reference book in the world and the only one in the United States covering all lines for the use of Federal, State, and City Government Departments, American and foreign consuls, exporters, merchants' and manufacturers' associations, boards of trade, libraries, banks, railroads, contractors, merchants, manufacturers, and buyers and sellers in all lines. The main portion of the text contains a list of manufacturers, arranged alphabetically by subject, State, city, and name and address of firm. There is also an index to American manufacturers, arranged alphabetically by name, which gives the home and branch offices, names of individual officers, etc., of all manufacturers having a capital investment of $50 000 or more, or a widely distributed business. A buyers' quick reference list of leading trade names, arranged alphabetically, is included, as well as an Appendix containing the names of architects, representative banks, and commercial organizations, all arranged alphabetically by State, city, and firm, and a list of trade publications arranged by subject. The Contents are: Section I, Finding List and Index; Section II, Lists of Manufacturers Classified According to Business; Section III, Manufacturers of the United States, Arranged Alphabetically by Name, Giving Home Offices, Branch Offices, Names of Officers, Sales Managers, Purchasing Agents, etc.; Section IV, Leading Trade Names, Brands, etc.; Section V, Appendix: Architects, Machinists, and Founders, Banks, Boards of Trade and Other Commercial Organizations, Leading Trade Papers, etc.

THE ELASTICITY AND RESISTANCE OF THE MATERIALS OF ENGINEERING.

By William H. Burr, M. Am. Soc. C. E. Seventh Edition, Thoroughly Revised. Cloth, 9¼ x 6¼ in., 19 + 928 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1915. $5.50.

The rapid development which has characterized all branches of engineering construction during the past decade carries with it, the author states, corresponding advances in experimental and analytical work in that field of engineering comprised in this book. In this new edition, prepared to meet the advancing requirements of the Profession, much of the older and obsolete material has been omitted, and many new topics have been added, the new subject-matter comprising, it is said, about three-fourths of the book. Among other new parts, the treatment of reinforced concrete, the general analysis of which as a development of the common theory of flexure was first given in a previous edition, has been extended, it is said, to cover practically all the principal features of that field. The analysis given is general, but simple and free, it is stated, from the modern complicated formulas, and the results of the most recent experimental investigations have been used for the requisite empirical data, in order, the author states, to make the book a real work on the Elasticity and Resistance of the Materials of Engineering, rather than a mere matter of Applied Mechanics. The Contents are: Part I, Analytical: Elementary Theory of Elasticity in Amorphous Solid Bodies; Flexure; Torsion; Hollow Cylinders and Spheres; Resilience; Combined Stress Conditions. Part II, Technical: Tension; Compression; Riveted Joints and Pin Connection; Long Columns; Shearing and Torsion; Bending or Flexure; Concrete-Steel Members; Rolled and Cast-Flanged Beams; Plate Girders; Miscellaneous Subjects; The Fatigue of Metals; The Flow of Solids. Appendix I, Elements of Theory of Elasticity in Amorphous Solid Bodies; General Equations; Thick Hollow Cylinders, and Spheres, and Torsion; Theory of Flexure. Appendix II, Clavarino's Formula. Appendix III, Resisting Capacity of Natural and Artificial Ice; Index.

FIELD ENGINEERING:

A Handbook of the Theory and Practice of Railway Surveying, Location and Construction. By William H. Searles, M. Am. Soc. C. E. Seventeenth Edition, Revised and Enlarged, by William H. Searles and Howard Chapin Ives, Assoc. M. Am. Soc. C. E. Volume I, Text, Volume II, Tables. Morocco, $6\frac{3}{4}$ x $4\frac{1}{4}$ in., illus., 2 v. in 1. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1915. $3.00.

The first edition of this book was issued in 1880. In this, the 1915, edition, the text has been entirely re-set, owing to the numerous changes found to be necessary to keep it up to date, resulting in the addition of about 150 pages of new matter and 16 new tables, the re-arrangement of certain portions, and the abridgment of others. The author's aim, as stated in the preface to the first edition, has been to present, in a progressive and logical manner, a complete handbook of railway engineering for field use, together with the problems, formulas, and tables best adapted to the needs of the field engineer and the engineering student. The Contents are: Vol. I, Text: Reconnaissance; Preliminary Survey; Theory of Maximum Economy in Grades and Curves; Location; Simple Curves; Compound Curves; Reversed Curves; Turnouts and Crossings; The Spiral Curve; Leveling; Cross-Sections; Calculation of Earthwork; Earthwork Tables; Earthwork Diagrams; Haul and Mass Diagram; Construction; Track Laying; Topographical Sketching; Adjustment of Instruments. Vol. II, Tables.

HANDBOOK OF CHEMISTRY AND PHYSICS:

A Ready-Reference Pocket Book of Chemical and Physical Data, Compiled from the Most Recent Authoritative Sources. Cloth, $6\frac{3}{4}$ x $4\frac{1}{2}$ in., 322 pp. Cleveland, Ohio, The Chemical Rubber Company, 1914. $2.00.

The aim of the publishers in issuing this book in its new and revised form, as stated in the preface, has been to present, in one compact volume, a comprehensive reference book on chemical and physical topics, for use in the laboratory and classroom. The material included in these pages is said to have been carefully selected from various standard sources, in accordance with the suggestions received from more than 1 000 members of high standing in the Chemical and Physical Profession. A complete bibliography of textbooks, manuals, reference books, and periodicals of interest to chemists and physicists is also included. The Chapter headings are: Antidotes for Poisons; General Chemical Tables; Properties of Matter; Heat; Hygrometric and Barometric Tables; Sound; Electricity and Magnetism; Light; Miscellaneous Tables; Definitions and Formulas; Laboratory Arts and Recipes; Measures and Units; Wire Tables; Mathematical Tables; Apparatus Lists; Bibliography; Problems; Index.

SIMPLIFIED REINFORCED CONCRETE MATHEMATICS:

Derivation of Simple Universal Formulas and Application of Same to Beams, Columns, and Arches, with Nomographic Computing Device. By Melvin D. Casler, Assoc. M. Am. Soc. C. E. Cloth, $7\frac{3}{4}$ x 5 in., illus., 6 + 66 pp. New York, D. Van Nostrand Company, 1915. $1.25.

The main purpose of this book, it is stated, is to provide the engineer with practical working formulas for the design and investigation of reinforced concrete members, and with means for applying these formulas with a minimum of computation. The proposed formulas, it is said, are derived for general application to beams subject to direct longitudinal stress in conjunction with transverse moment, to eccentrically loaded columns, and to arches. One of the objects of the book, as stated by the author, is to simplify the formulas and their application to beams, columns and arches, without loss in mathematical accuracy, so as to make the use of special curves and tables for various assumptions to properties, stresses, dimensions, etc., etc. The author has also included some labor-saving devices for use in proportioning members and has demonstrated, by definite examples, the application of the formulas to beams, columns, and arches. For work of varied nature, the methods given in the book are said to effect a large saving in time over prevalent methods of computation. The Contents are: Derivation of Formulas; Labor-Saving Devices; Illustrative Examples; General Notes on Reinforced Concrete Design.

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Reinforced Concrete in Practice: A Textbook for Those Engaged Upon Structural Works. By A. Alban H. Scott. New York and London, 1915.

American Society for Testing Materials Affiliated with the International Association for Testing Materials: Year Book, 1915. Containing the Standards and Tentative Standards. Edited by the Secretary-Treasurer. Philadelphia, 1915.

Public Utilities Reports Annotated: Containing Decisions of the Public Service Commissions and of State and Federal Courts ; C and D. Rochester and New York, 1915.

The A B C of Iron and Steel, With a Directory of the Iron and Steel Works and Their Products of the United States and Canada. Edited by A. O. Backert. Cleveland, Ohio, 1915.

Supplement au Manuel Hydrologique du Bassin de la Seine. Ministere des Travaux Publics, Ponts et Chaussees, Direction des Routes et de la Navigation. Paris, 1909.

Grinding Machinery. By James J. Guest. London, 1915.

Producer Gas. By J. Emerson Dowson and A. T. Larter. Third Edition. New York and London, 1912.

The Electric Railway. By A. Morris Buck. New York and London, 1915.

The Chemistry of Gas Manufacture: A Practical Handbook on the Production, Purification, and Testing of Illuminating and Fuel Gas and on the Bye-Products of Gas Manufacture ; Vol. I, Materials and Processes. By W. J. Atkinson Butterfield. Fourth Edition. London, 1907.

SUMMARY OF ACCESSIONS

(From October 4th to November 1st, 1915)

Donations (including 28 duplicates)	411
By purchase	10
Total	421

MEMBERSHIP

(From October 8th to November 4th, 1915)

ADDITIONS

MEMBERS

		Date of Membership.
CLAPP, FRANK LEMUEL. 195 Boston St., Boston, Mass	Assoc. M. M.	May 4, 1909 Sept. 21, 1915
FORTIER, SAMUEL. Chf. of Irrig. Investigations, U. S. Dept. of Agriculture, Washington, D. C.		Aug. 31, 1915
OLDER, CLIFFORD. Bridge Engr., Ill. Highway Comm., Springfield, Ill	Assoc. M. M.	Oct. 4, 1905 July 7, 1915
PRELL, LOUIS HENRY. In Chg. of Constr., Dam No. 35, Ohio River, New Richmond, Ohio		Aug. 31, 1915
WITT, JACKSON FRANKLIN. County Engr., Dallas County, Dallas, Tex	Assoc. M. M.	Oct. 31, 1911 Sept. 21, 1915
WYNNE-ROBERTS, ROBERT OWEN. Cons. Engr., 310 Temple Bldg., Toronto, Ont., Canada		June 3, 1915

ASSOCIATE MEMBERS

ARCHIBALD, EDMUND OLIVER. Asst. Engr., Alaska Eng. Comm., Anchorage, Alaska	Aug. 31, 1915
CASE, CHARLES ANDREW. City Engr., Neodesha, Kans	Aug. 31, 1915
CURRAN, THOMAS FRANCIS. Deputy City Engr., 50 Margaret St., Utica, N. Y	Aug. 31, 1915
McFARLAND, HARRY FONTAINE, JR. Asst. Engr. to Engr. of Bridges, St. L. & San Francisco R. R., 627 Frisco Bldg., St. Louis, Mo	Aug. 31, 1915
SHAE, SHAO-YING DOUGLAS. Care, Canton-Hankow Ry., Yochow City, Hunan, China	June 3, 1915

JUNIORS

ANDERSON, JOHN EDWARD. Care, Cox & Co., 16 Charing Cross, London, S. W., England	Aug. 31, 1915
BROWN, HARRY MADARA. 1531 Park Rd., Washington, D. C.	June 3, 1915
LEHMAN, HENRY MARON. 50 Court St., Brooklyn, N. Y	June 3, 1915
MARX, RAYMOND. Asst. Supt. of Constr., Bing & Bing Constr. Co., 119 West 40th St. (Res., 645 West End Ave.), New York City	Aug. 31, 1915
MUNSON, CHARLES HAROLD. 1360A Scenic Ave., Berkeley, Cal	Aug. 31, 1915

CHANGES OF ADDRESS

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POMMERER, ROBERT WILLIAM. Realty Bldg., White Plains, N. Y.

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JUNIORS

ALGER, RALPH TISDALE. Res. Engr., N. Y. Continental Jewell Filtration Co., Shelbyville, Ky.

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GOWEN, JOHN FELLOWS. 550 Palisade Ave., Yonkers, N. Y.

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RESIGNATIONS

ASSOCIATE MEMBERS	Date of Resignation.
Herring, Jerome Campbell	Sept. 20, 1915
Taylor, Oliver Kirk, Jr	Sept. 20, 1915

DEATHS

Berquist, Axel Samuel Frederick. Elected Member, June 6th, 1906; died October 6th, 1915.

Bowen, Walter Cox. Elected Junior, December 31st, 1913; died May 8th, 1915.

Coit, Edward Woolsey. Elected Fellow, September 20th, 1872; died September 25th, 1915.

Cole, Osman Fred. Elected Associate Member, September 2d, 1908; died September 27th, 1915.

Du Bois, Augustus Jay. Elected Junior, July 7th, 1875; Member, October 5th, 1892; died October 19th, 1915.

GRAY, EDWARD. Elected Associate Member, May 1st, 1907; Member, December 6th, 1910; died October 2d, 1915.

HARTRICK, EDWARD MACAULAY. Elected Member, February 1st, 1899; died August, 1915.

HOVENDEN, THOMAS. Elected Associate Member, July 9th, 1912; died September 19th, 1915.

KING, WILLIAM BYRD. Elected Member, October 7th, 1896; died October 11th, 1915.

PIHL, OLAF RIDLEY. Elected Member, October 2d, 1899; died October 14th, 1915.

Total Membership of the Society, November 4th, 1915, 7 886.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(October 4th to November 1st, 1915)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., St. Louis, Mo., 30c.
(2) *Proceedings*, Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur*, München, Germany.
(8) *Stevens Institute Indicator*, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, 432 Fourth Ave., New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *Engineering Record*, New York City, 10c.
(15) *Railway Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway Review*, Chicago, Ill., 15c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 20c.
(21) *Railway Engineer*, London, England, 1s. 2d.
(22) *Iron and Coal Trades Review*, London, England, 6d.
(23) *Railway Gazette*, London, England, 6d.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *Railway Age Gazette*, Mechanical Edition, New York City, 20c.
(26) *Electrical Review*, London, England, 4d.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéclales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *Cornell Civil Engineer*, Ithaca, N. Y.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Technisches Gemeindeblatt*, Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung*, Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift*, Berlin, Germany.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Colliery Engineer*, Scranton, Pa., 25c.
(46) *Scientific American*, New York City, 15c.
(47) *Mechanical Engineer*, Manchester, England, 3d.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia, 25 kop.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria, 70h.
(54) *Transactions*, Am. Soc. C. E., New York City, $12.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $6.

(57) *Colliery Guardian*, London, England, 5d.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings*, American Water-Works Assoc., Troy, N. Y.
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CHEMI-HYDROMETRY AND ITS APPLICATION TO THE PRECISE TESTING OF HYDRO-ELECTRIC GENERATORS

By Benjamin F. Groat, M. Am. Soc. C. E.

To be Presented December 15th, 1915.

Synopsis.

This paper is divided into five parts. A summary of each part (except Part IV) will be found on pages 2107, 2158, 2288, and 2383.

Chemi-hydrometry, more explicitly defined in Section 6, relates to the measurement of quantities of fluids by chemical methods.

Introduction.

Owing to the large bonus and liquidated damages clauses of the contract considered in this paper, it was apparent that unusual precision was necessary in executing the tests to determine the performance and efficiency of the new hydro-electric equipment. Besides this, there were other equally cogent reasons for making the tests as precise as possible with such methods as sound engineering practice and field methods could furnish. Among these may be mentioned the strong probability of large future purchases of hydro-electric equipment, and the concomitant necessity for accurate methods of testing, coupled with the fact that there had not yet been developed any method for measuring the volume of large quantities of turbulently

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flowing water, which would furnish convincing results reliable to within 1% of the true quantity.

The uncertainty attending the measurement of large volumes of flowing water by methods heretofore used, under the most favorable circumstances and according to the best information at hand, is about as follows:

Moving screen.......	½ to 1%
Weirs	1%
Current meters.......	2 to 8%, and even higher.

The last case supposes that the meters have been rated in the usual manner in still water, and that this rating is presumed to remain unchanged during the measurements. Current meters under these circumstances have been known by the writer to be in error by more than 100% of the true velocity.

So far as the writer is aware, his tests of turbines in 1911* were the first in which any attempt was ever made to correct the still-water ratings of current meters. During these tests it was shown that screw meters under-register and cup meters over-register, and the discrepancy thus arising was found to be 8% as an average, the particular types of meter involved being the larger sizes of the Haskell and Price patterns. At certain points in the races corrections exceeding 15% were found to be necessary.

In view of these facts, and the fact that the contract under consideration specified the Haskell meter as one of the instruments for testing, it was decided to use the chemical method in parallel with three types of meter: the Haskell, Ott, and small Price. The Ott meter is of the screw type, and the small Price is a more accurate instrument than the larger type.

Ratings of all the meters were made at the Naval Tank at the University of Michigan, and curves were constructed showing the relative divergences of the ratings for given amounts of oscillation which were supposed to represent disturbance or "turbulence" of flow. Both transverse and longitudinal oscillations, with reference to the direction of motion of the meters, or water, were studied.

The results of the subsequent tests on the Allis-Chalmers equipment in the summer of 1914 confirm the conclusions drawn from the tests of 1911 concerning current meters, and the chemical tests, which

* *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 819.

are hereafter shown to be accurate to a fraction of 1%, prove conclusively that current-meter work in perturbed water must include corrections to be applied to the still-water rating, if accurate results are to be exacted. They also show that the methods of correcting the meter records used by the writer in 1914 are sufficiently accurate to secure results within 1 or 2%, even where corrections of 15%, or more, must be applied to records based on still-water ratings.

The chemical method had never been used in measuring large quantities of water with precision, the quantities dealt with usually being limited to a few cubic feet per second in the case of high-head plants where the turbines were fed by pipe lines. Somewhat larger volumes had been measured by Dr. Leon W. Collet, in the case of Swiss mountain torrents, and Stromeyer, in England, had measured boiler feed and some small streams at even earlier dates.

After reviewing these experiments, a plan was devised for testing large-capacity turbine units where discharges of several thousand cubic feet per second could be measured with precision. In devising this plan, the very accurate method of Dr. R. Mellet, of the University of Lausanne, Switzerland, for performing high-precision titrations was adopted, but it was found on trial that the method of unbalanced evaporations for conducting the remainder of the tests introduced a systematic error of greater or less degree, the magnitude of which in the present series of tests was about $\frac{1}{3}$ of 1 per cent.

The writer was not enthusiastic over the chemical method until he read Dr. Mellet's extremely valuable paper, presenting the basis for these exceedingly accurate titrations, and demonstrating clearly what degree of accuracy can be secured by such a procedure. Even then it was questionable as to how applicable the method would be to turbines of large capacity operating on low heads with correspondingly large discharges. The very satisfactory tests cited by Dr. Mellet, where the discharge of the Day power-plant at Vallorbe was accurately determined, would scarcely give any idea of the possible precision attainable in the proposed tests, because the discharge measured at Vallorbe was only 262 liters per sec., whereas it would now be necessary to measure 50 000 liters per sec. with all possible precision.

Especially was there much doubt as to the uniformity of mixture which could be secured, and as to the method of introducing the chemical into the head-race. The methods of sampling were also to

be determined, as well as the size of the solution tank and the proportions of the other parts of the testing equipment.

After some further study of actual tests, it was found that the method suggested by the writer in a paper which he presented before the Engineers' Society of Western Pennsylvania in December, 1913, eliminated the error above mentioned, along with a number of other errors, and that, in fact, it supplies all the elements of precision necessary for the most refined work. The formula which indicates the methods first appears in a somewhat more complicated form than necessary.* These methods have since been applied successfully by the writer in several tests in different parts of the country.

In order to show that turbine testing has been anything but an exact science in the past, it may be stated that there have been several extensive series of tests which can be regarded in no other light than as failures, in so far as decisively definite results are concerned, and that in at least one case, after an expenditure of many thousands of dollars, there was an uncertainty as to the resulting efficiency amounting to 3 per cent. Current meters were used in that test.

Turbine testing and discharge gauging have been more of an art than a science. Results have depended more for accuracy on the particular person who directed the work than on any method heretofore used. It has been a common occurrence that there has been a complete failure to interpret the data correctly, with the result that much of the work in discharge measurements has been unsatisfactory. When results have been shown to be accurate, we may rest assured that the person conducting the tests has arrived at a conclusion only by the most painstaking efforts and perseverance.

The results of the studies mentioned will be treated in five parts:

I.—History and Theory;
II.—Errors;
III.—Description of Testing Plant;
IV.—The Tests;
V.—Current Meters.

A brief summary is given at the beginning of each part.

* *Proceedings*, Engineers' Society of Western Pennsylvania, May, 1914, Vol. 30, p. 374.

PART I.

HISTORY AND THEORY.

Summary.

1.—In Part I a complete theory of mixtures of salt solutions of various densities has been evolved and elaborated into a comprehensive treatise on the subject. This includes tables and diagrams of properties of the salt solution. This theory is necessary for high-precision work on account of the shrinkage in volume which takes place when two or more different salt solutions are mixed.

The term "Chemi-hydrometry" has been proposed as the name for this new branch of engineering science.

The reader's attention is called to the important fact that there is great advantage to be gained by measuring quantities of the solutions by weight rather than by volume. This eliminates in large measure the necessity for observing the temperatures of the solutions and making the corrections therefor. There is further gain in accuracy due to the fact that there is less likelihood of contaminating the samples by reason of neglect to clean the thermometer each time before it is dipped into a sample or other solution. Moreover, this method gives the real quantity which is desired in turbine testing, namely, the weight of the water passing through the water-wheels as against the volume.

A complete method has been devised for the precise testing of hydro-electric units of large capacity with a chemical injected into and mixed with the feed-water. This method is based on the original suggestion by Schloesing in France, in 1863, the method of precise titrations of weak salt solutions devised by Dr. R. Mellet, and the design and operation of chemical testing plants, as suggested in a paper by the writer before the Engineers' Society of Western Pennsylvania in December, 1913.* This paper also contains the formula which leads to the method of balanced evaporations, accomplished by special dilutions in the laboratory.

The formula for balanced evaporations is treated in full, as it applies to tests under various conditions. It leads to the elimination of all uncertainty as to the properties of the normal head-water. It indi-

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* *Proceedings,* Engineers' Society of Western Pennsylvania, May, 1914.

cates and represents a definite laboratory procedure which has been outlined in accordance therewith.

The coefficient of shrinkage of volume, when two salt solutions are mixed, has been fully treated and tabulated.

Theoretically correct equations, involving concentrations only, are established.

2.—Theory and Properties of the Salt Solution.—We are indebted to Landolt and Börnstein* for the most complete tables of the densities of salt solutions of different percentages up to saturated solutions at temperatures varying from 0° to 80° cent. These tables, therefore, will be taken as the basis for the development of the theory of the salt solution as applied in the chemical method for measuring quantities of water.

As the theory which is to be presented concerns largely the concentrations of various solutions, it will be convenient to elaborate on the tables to some extent by appending a column of concentrations. It will frequently be convenient to know also the quantity, either by volume or by weight, of distilled water or of salt, contained in a solution of given concentration. Therefore, additional columns, for these and similar purposes, are appended, so as to furnish the data in Table 1, which data are limited to temperatures between 0° and 40° cent.

3.—Approximate Formulas for Weak Solutions.—As Table 1 does not extend to salt solutions of less than ½ of 1%, several equations have been devised for computing densities in terms of concentrations, or percentages, and *vice versa.* These equations are merely approximate, but cannot be far from correct, especially as regards relative errors in computing densities. The general equation for density, d_x, at temperature, x, in terms of the density, d, of distilled water, at temperature, x, and one parameter, n, depending on the temperature, is,

$$d_x = d + n_x p_x \left\{ \begin{array}{c} \text{Limited to solutions of less than } \tfrac{1}{2}\% \\ \text{in salt} \end{array} \right\} \ldots\ldots\ldots\ldots (1)$$

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* "Physikalisch-Chemische Tabellen." Landolt-Börnstein. Published by Julius Springer, Berlin, 1912, p. 260. See also a paper by W. J. Walker "On the Relationship between the Viscosity, Density, and Temperature of Salt Solutions." *Phil. Mag.*, February, 1914, Series 6, Vol. 27, No. 158.

TABLE 1.—ONE LITER OF SALT SOLUTION, OF VARYING DENSITY, AT DIFFERENT TEMPERATURES.

TEMPERATURE = 0° CENT.

Volume of 1 Gramme of Distilled Water = 1.000132 c.c.

Percentage of salt solution.	Density, in grammes per cubic centimeter.	Concentration, in grammes per liter.	Weight of distilled water per liter of solution, in grammes.	Volume of distilled water per liter of solution, in cubic centimeters.	Apparent volume of salt per liter of solution, in cubic centimeters.	Salt per cubic centimeter of apparent volume, in grammes.	Apparent volume occupied per gramme of salt, in cubic centimeters.
0.5	1.0037	5.0185	998.68	998.81	1.1867	4.2289	0.23646
1.0	1.0076	10.076	997.52	997.66	2.3443	4.2981	0.23266
1.5	1.0114	15.171	996.23	996.36	3.6395	4.1684	0.23990
2.0	1.0153	20.306	994.99	995.13	4.8747	4.1656	0.24006
2.5	1.0191	25.478	993.62	993.75	6.2468	4.0786	0.24519
3.0	1.0230	30.690	992.31	992.44	7.5590	4.0601	0.24630
3.5	1.0268	35.938	990.86	990.99	9.0072	3.9899	0.25063
4.0	1.0307	41.228	989.47	989.60	10.397	3.9654	0.25218
4.5	1.0345	46.552	987.95	988.08	11.922	3.9047	0.25610
5.0	1.0384	51.920	986.48	986.61	13.390	3.8775	0.25789
5.5	1.0422	57.321	984.88	985.01	14.991	3.8237	0.26153
6.0	1.0461	62.766	983.33	983.46	16.536	3.7957	0.26345
6.5	1.0500	68.250	981.75	981.88	18.120	3.7666	0.26549
7.0	1.0538	73.766	980.03	980.16	19.837	3.7186	0.26892
7.5	1.0577	79.328	978.37	978.50	21.499	3.6898	0.27101
8.0	1.0616	84.928	976.67	976.80	23.199	3.6608	0.27316
8.5	1.0654	90.559	974.84	974.97	25.030	3.6180	0.27639
9.0	1.0693	96.237	973.06	973.19	26.809	3.5897	0.27857
9.5	1.0732	101.95	971.25	971.38	28.621	3.5621	0.28073
10.0	1.0771	107.71	969.39	969.52	30.482	3.5336	0.28300
10.5	1.0810	113.50	967.50	967.63	32.372	3.5061	0.28521
11.0	1.0849	119.34	965.56	965.69	34.313	3.4780	0.28752
11.5	1.0888	125.21	963.59	963.72	36.283	3.4509	0.28977
12.0	1.0927	131.12	961.58	961.71	38.293	3.4241	0.29204
12.5	1.0966	137.07	959.53	959.66	40.343	3.3976	0.29432
13.0	1.1005	143.06	957.44	957.57	42.434	3.3714	0.29661
13.5	1.1044	149.09	955.31	955.44	44.564	3.3455	0.29890
14.0	1.1083	155.16	953.14	953.27	46.734	3.3201	0.30120
14.5	1.1123	161.28	951.02	951.15	48.854	3.3013	0.30291
15.0	1.1162	167.43	948.77	948.90	51.105	3.2762	0.30523
15.5	1.1202	173.63	946.57	946.69	53.305	3.2573	0.30700
16.0	1.1241	179.86	944.24	944.36	55.635	3.2329	0.30932
16.5	1.1281	186.14	941.96	942.08	57.916	3.2140	0.31114
17.0	1.1321	192.46	939.64	939.76	60.236	3.1951	0.31298
17.5	1.1361	198.82	937.28	937.40	62.596	3.1762	0.31483
18.0	1.1401	205.22	934.88	935.00	64.997	3.1574	0.31672
18.5	1.1441	211.66	932.44	932.56	67.437	3.1386	0.31861
19.0	1.1481	218.14	929.96	930.08	69.917	3.1200	0.32051
19.5	1.1521	224.66	927.44	927.56	72.438	3.1014	0.32243
20.0	1.1562	231.24	924.96	925.08	74.918	3.0866	0.32398
20.5	1.1602	237.84	922.36	922.48	77.518	3.0682	0.32592
21.0	1.1643	244.50	919.80	919.92	80.079	3.0532	0.32752
21.5	1.1684	251.21	917.19	917.31	82.689	3.0380	0.32916
22.0	1.1724	257.93	914.47	914.59	85.409	3.0199	0.33113
22.5	1.1765	264.71	911.79	911.91	88.090	3.0050	0.33278
23.0	1.1806	271.54	909.06	909.18	90.820	2.9899	0.33446
23.5	1.1848	278.43	906.37	906.49	93.510	2.9775	0.33585
24.0	1.1889	285.34	903.56	903.68	96.321	2.9624	0.33756
25.0	1.1972	299.30	897.90	898.02	101.98	2.9348	0.34073
26.0	1.2056	313.46	892.14	892.26	107.74	2.9093	0.34371
26.4	1.2089	319.15	889.75	889.87	110.13	2.8979	0.34507

TABLE 1.—(*Continued.*)

TEMPERATURE = 10° CENT.

Volume of 1 Gramme of Distilled Water = 1.000273 c.c.

Percentage of salt solution.	Density, in grammes per cubic centimeter.	Concentration. in grammes per liter.	Weight of distilled water per liter of solution, in grammes.	Volume of distilled water per liter of solution, in cubic centimeters.	Apparent volume of salt per liter of solution, in cubic centimeters.	Salt per cubic centimeter of apparent volume, in grammes.	Apparent volume occupied per gramme of salt, in cubic centimeters.
0.5	1.0034	5.0170	998.38	998.66	1.3444	3.7317	0.26797
1.0	1.0071	10.071	997.03	997.30	2.6988	3.7316	0.26798
1.5	1.0108	15.162	995.64	995.91	4.0902	3.7069	0.26977
2.0	1.0145	20.290	994.21	994.48	5.5186	3.6766	0.27199
2.5	1.0182	25.455	992.74	993.01	6.9840	3.6447	0.27437
3.0	1.0219	30.657	991.24	991.51	8.4864	3.6125	0.27682
3.5	1.0256	35.896	989.70	989.97	10.026	3.5802	0.27931
4.0	1.0293	41.172	988.13	988.40	11.602	3.5486	0.28179
4.5	1.0330	46.485	986.52	986.78	13.216	3.5173	0.28431
5.0	1.0367	51.835	984.86	985.13	14.866	3.4868	0.28679
5.5	1.0405	57.228	983.27	983.54	16.460	3.4767	0.28762
6.0	1.0442	62.652	981.55	981.82	18.184	3.4454	0.29024
6.5	1.0479	68.114	979.79	980.05	19.947	3.4147	0.29285
7.0	1.0517	73.619	978.08	978.35	21.652	3.4001	0.29411
7.5	1.0554	79.155	976.24	976.51	23.488	3.3700	0.29673
8.0	1.0592	84.736	974.46	974.73	25.270	3.3532	0.29822
8.5	1.0629	90.346	972.55	972.82	27.180	3.3240	0.30084
9.0	1.0667	96.003	970.70	970.96	29.038	3.3061	0.30247
9.5	1.0705	101.70	968.80	969.06	30.936	3.2874	0.30419
10.0	1.0742	107.42	966.78	967.04	32.956	3.2595	0.30680
10.5	1.0780	113.19	964.81	965.07	34.927	3.2408	0.30857
11.0	1.0818	119.00	962.80	963.06	36.937	3.2217	0.31039
11.5	1.0856	124.84	960.76	961.02	38.978	3.2028	0.31222
12.0	1.0894	130.73	958.67	958.93	41.068	3.1833	0.31414
12.5	1.0933	136.66	956.64	956.90	43.099	3.1708	0.31537
13.0	1.0971	142.62	954.48	954.74	45.259	3.1512	0.31734
13.5	1.1009	148.62	952.28	952.54	47.460	3.1315	0.31934
14.0	1.1048	154.67	950.13	950.39	49.611	3.1177	0.32075
14.5	1.1087	160.70	948.00	948.26	51.741	3.1058	0.32197
15.0	1.1125	166.88	945.62	945.88	54.122	3.0834	0.32432
15.5	1.1164	173.04	943.36	943.62	56.382	3.0691	0.32583
16.0	1.1203	179.25	941.05	941.31	58.693	3.0540	0.32744
16.5	1.1242	185.49	938.71	938.97	61.034	3.0391	0.32904
17.0	1.1281	191.78	936.32	936.58	63.424	3.0238	0.33071
17.5	1.1321	198.12	933.98	934.23	65.765	3.0125	0.33195
18.0	1.1360	204.48	931.52	931.77	68.226	2.9971	0.33366
18.5	1.1400	210.90	929.10	929.35	70.646	2.9853	0.33497
19.0	1.1439	217.34	926.56	926.81	73.187	2.9697	0.33674
19.5	1.1479	223.84	924.06	924.31	75.688	2.9574	0.33813
20.0	1.1519	230.38	921.52	921.77	78.228	2.9450	0.33956
20.5	1.1559	236.96	918.94	919.19	80.809	2.9323	0.34102
21.0	1.1600	243.60	916.40	916.65	83.350	2.9226	0.34216
21.5	1.1640	250.26	913.74	913.99	86.011	2.9096	0.34369
22.0	1.1681	256.98	911.12	911.37	88.631	2.8994	0.34489
22.5	1.1721	263.72	908.38	908.63	91.372	2.8862	0.34647
23.0	1.1762	270.53	905.67	905.92	94.083	2.8754	0.34777
23.5	1.1803	277.37	902.93	903.18	96.824	2.8647	0.34908
24.0	1.1845	284.28	900.22	900.47	99.534	2.8561	0.35013
25.0	1.1927	298.18	894.52	894.76	105.24	2.8333	0.35294
26.0	1.2011	312.29	888.81	889.05	110.95	2.8147	0.35528
26.4	1.2045	317.99	886.51	886.75	113.25	2.8079	0.35614

TABLE 1.—(*Continued.*)

TEMPERATURE = 15° CENT.

Volume of 1 Gramme of Distilled Water = 1.000874 c.c

Percentage of salt solution.	Density, in grammes per cubic centimeter.	Concentration, in grammes per liter.	Weight of distilled water per liter of solution, in grammes.	Volume of distilled water per liter of solution, in cubic centimeters.	Apparent volume of salt per liter of solution, in cubic centimeters.	Salt per cubic centimeter of apparent volume, in grammes.	Apparent volume occupied per gramme of salt, in cubic centimeters.
0.5	1.0028	5.0140	997.79	998.66	1.3419	3.7365	0.26763
1.0	1.0064	10.064	996.34	997 21	2.7932	3.6030	0.27754
1.5	1.0100	15.150	994.85	995.72	4.2805	3.5393	0.28254
2.0	1.0137	20.274	993.43	994.29	5.7058	3.5532	0.28143
2.5	1.0173	25.432	991.87	992.73	7.2651	3.5006	0.28567
3.0	1.0209	30.627	990.27	991.12	8.8625	3.4558	0.28937
3.5	1.0246	35.861	988 74	989.60	10.397	3.4492	0.28992
4.0	1.0282	41.128	987.07	987.93	12.065	3.4089	0.29335
4.5	1.0319	46.436	985.46	986.33	13.675	3.3956	0.29449
5.0	1.0355	51.775	983.72	984.58	15.415	3.3587	0.29773
5.5	1.0392	57.156	982.04	982.90	17.098	3.3428	0.29915
6.0	1.0429	62.574	980.33	981.18	18.817	3.3254	0.30072
6.5	1.0466	68.029	978 57	979.43	20.574	3.3066	0.30243
7.0	1.0503	73.521	976.78	977.63	22.367	3.2870	0.30423
7.5	1.0540	79.050	974.95	975.80	24.198	3.2668	0.30611
8.0	1.0577	84.616	973.08	973.93	26.066	3.2462	0.30805
8.5	1.0614	90.219	971.18	972.03	27.970	3.2256	0.31002
9.0	1.0651	95.859	969 24	970.09	29.912	3.2047	0.31204
9.5	1.0688	101.54	967 26	968.11	31.895	3.1836	0.31411
10.0	1.0726	107.26	965.34	966.18	33.816	3.1719	0.31527
10.5	1.0763	113.01	963.29	964.13	35.868	3.1507	0.31739
11.0	1.0801	118.81	961.29	962.13	37.870	3.1373	0.31874
11.5	1.0838	124.64	959.16	960.00	40.002	3.1158	0.32094
12 0	1.0876	130.51	957.09	957.93	42.074	3.1019	0.32238
12.5	1.0914	136.42	954.98	955.81	44.185	3.0875	0.32389
13.0	1.0952	142.38	952.82	953.65	46.347	3.0720	0.32552
13.5	1.0990	148.36	950.64	951.47	48.529	3.0571	0.32710
14.0	1.1028	154.39	948.41	949.24	50.761	3.0415	0.32878
14.5	1.1066	160.46	946.14	946.97	53.033	3.0256	0.33051
15.0	1.1105	166.58	943.92	944.74	55.255	3.0147	0.33170
15.5	1.1143	172.72	941.58	942.40	57.597	2.9988	0.33347
16.0	1.1182	178.91	939.29	940.11	59.889	2.9874	0.33474
16.5	1.1221	185.15	936.95	937.77	62.231	2.9752	0.33611
17.0	1.1260	191.42	934.58	935.40	64.603	2.9630	0.33749
17.5	1.1299	197.73	932.17	932.98	67.015	2.9505	0.33892
18.0	1.1338	204.08	929.72	930.53	69.467	2.9378	0.34039
18.5	1.1378	210.49	927.31	928.12	71.880	2.9284	0.34149
19.0	1.1417	216.92	924.78	925.59	74.412	2.9150	0.34304
19.5	1.1457	223.41	922.29	923.10	76.904	2.9050	0.34423
20.0	1.1497	229.94	919.76	920.56	79.436	2.8947	0.34546
20.5	1.1537	236 51	917.19	917.99	82.008	2.8840	0.34674
21.0	1.1577	243.12	914.58	915.38	84.621	2.8730	0.34806
21.5	1.1617	249.77	911.93	912.73	87.273	2.8619	0.34941
22.0	1.1657	256.45	909.25	910.04	89.955	2.8509	0.35077
22.5	1.1698	263.20	906.60	907.39	92.608	2.8421	0.35185
23.0	1.1739	270.00	903.90	904.69	95.310	2.8329	0.35300
23.5	1.1780	276.83	901.17	901.96	98.042	2.8236	0.35416
24.0	1.1821	283.70	898.40	899.19	100.81	2.8142	0.35534
25.0	1.1904	297.60	892.80	893.58	106.42	2.7965	0.35759
26.0	1.1987	311.66	887.04	887.82	112.18	2.7782	0.35994
26.4	1.2021	317.35	884.75	885.52	114.48	2.7721	0.36074
26.8	1.2055	323.07	882.43	883.20	116.80	2.7660	0.36153

TABLE 1.—(*Continued.*)

TEMPERATURE = 20° CENT.

Volume of 1 Gramme of Distilled Water = 1.001773 c.c.

Percentage of salt solution.	Density, in grammes per cubic centimeter.	Concentration, in grammes per liter.	Weight of distilled water per liter of solution, in grammes.	Volume of distilled water per liter of solution, in cubic centimeters.	Apparent volume of salt per liter of solution, in cubic centimeters.	Salt per cubic centimeter of apparent volume, in grammes.	Apparent volume occupied per gramme of salt, in cubic centimeters.
0.5	1.0019	5.0095	996.89	998.66	1.3420	3.7329	0.26789
1.0	1.0054	10.054	995.35	997.11	2.8893	3.4797	0.28738
1.5	1.0090	15.135	993.86	995.62	4.3729	3.4611	0.28893
2.0	1.0126	20.252	992.35	994.11	5.8926	3.4369	0.29096
2.5	1.0161	25.402	990.70	992.45	7.5455	3.3665	0.29704
3.0	1.0197	30.591	989.11	990.86	9.1373	3.3479	0.29869
3.5	1.0233	35.816	987.48	989.23	10.765	3.3271	0.30056
4.0	1.0269	41.076	985.82	987.57	12.428	3.3051	0.30256
4.5	1.0305	46.372	984.13	985.87	14.127	3.2825	0.30465
5.0	1.0341	51.705	982.40	984.14	15.863	3.2595	0.30680
5.5	1.0378	57.079	980.72	982.46	17.540	3.2542	0.30729
6.0	1.0414	62.484	978.92	980.65	19.348	3.2295	0.30965
6.5	1.0450	67.925	977.08	978.81	21.193	3.2051	0.31201
7.0	1.0487	73.409	975.29	977.02	22.980	3.1945	0.31304
7.5	1.0523	78.922	973.38	975.10	24.896	3.1701	0.31545
8.0	1.0560	84.480	971.52	973.24	26.756	3.1574	0.31671
8.5	1.0597	90.074	969.63	971.35	28.655	3.1434	0.31813
9.0	1.0633	95.697	967.60	969.32	30.681	3.1191	0.32061
9.5	1.0670	101.36	965.64	967.35	32.648	3.1046	0.32210
10.0	1.0707	107.07	963.63	965.34	34.661	3.0891	0.32372
10.5	1.0744	112.81	961.59	963.29	36.705	3.0734	0.32537
11.0	1.0781	118.59	959.51	961.21	38.789	3.0573	0.32708
11.5	1.0819	124.42	957.48	959.18	40.822	3.0479	0.32810
12.0	1.0856	130.27	955.33	957.02	42.976	3.0312	0.32990
12.5	1.0894	136.18	953.22	954.91	45.090	3.0202	0.33111
13.0	1.0931	142.10	951.00	952.69	47.314	3.0033	0.33296
13.5	1.0969	148.08	948.82	950.50	49.498	2.9916	0.33427
14.0	1.1007	154.10	946.60	948.28	51.722	2.9794	0.33564
14.5	1.1045	160.15	944.35	946.02	53.976	2.9671	0.33703
15.0	1.1083	166.24	942.06	943.73	56.270	2.9543	0.33849
15.5	1.1122	172.39	939.81	941.48	58.524	2.9456	0.33949
16.0	1.1160	178.56	937.44	939.10	60.898	2.9321	0.34105
16.5	1.1199	184.78	935.12	936.78	63.222	2.9227	0.34215
17.0	1.1237	191.03	932.67	934.32	65.676	2.9087	0.34380
17.5	1.1276	197.33	930.27	931.92	68.081	2.8985	0.34501
18.0	1.1315	203.67	927.83	929.48	70.525	2.8879	0.34627
18.5	1.1355	210.07	925.43	927.07	72.929	2.8805	0.34717
19.0	1.1394	216.49	922.91	924.55	75.454	2.8692	0.34853
19.5	1.1433	222.94	920.36	921.99	78.008	2.8579	0.34991
20.0	1.1473	229.46	917.84	919.47	80.533	2.8493	0.35097
20.5	1.1513	236.02	915.28	916.90	83.097	2.8403	0.35208
21.0	1.1553	242.61	912.69	914.31	85.692	2.8312	0.35321
21.5	1.1593	249.25	910.05	911.66	88.336	2.8216	0.35441
22.0	1.1633	255.93	907.37	908.98	91.021	2.8118	0.35565
22.5	1.1674	262.66	904.74	906.34	93.656	2.8045	0.35657
23.0	1.1714	269.42	901.98	903.58	96.421	2.7942	0.35788
23.5	1.1755	276.24	899.26	900.85	99.146	2.7862	0.35891
24.0	1.1796	283.10	896.50	898.09	101.91	2.7779	0.35998
25.0	1.1879	296.98	890.92	892.50	107.50	2.7626	0.36198
26.0	1.1963	311.04	885.26	886.83	113.17	2.7484	0.36384
26.4	1.1996	316.69	882.91	884.48	115.52	2.7414	0.36477
26.8	1.2030	322.40	880.60	882.16	117.84	2.7359	0.36551

TABLE 1.—(*Continued.*)

TEMPERATURE = 40° CENT.

Volume of 1 Gramme of Distilled Water = 1.00782 c.c.

Percentage of salt solution.	Density, in grammes per cubic centimeter.	Concentration, in grammes per liter.	Weight of distilled water per liter of solution, in grammes.	Volume of distilled water per liter of solution, in cubic centimeters.	Apparent volume of salt per liter of solution, in cubic centimeters.	Salt per cubic centimeter of apparent volume, in grammes.	Apparent volume occupied per gramme of salt, in cubic centimeters.
0.5	0.9957	4.9785	990.72	998.47	1.5311	3.2516	0.30754
1.0	0.9991	9.991	989.11	996.84	3.1562	3.1655	0.31590
1.5	1.0025	15.038	987.46	995.18	4.8161	3.1224	0.32026
2.0	1.0060	20.120	985.88	993.59	6.4104	3.1386	0.31861
2.5	1.0094	25.235	984.16	991.86	8.1388	3.1006	0.32252
3.0	1.0129	30.387	982.51	990.20	9.8038	3.0995	0.32263
3.5	1.0163	35.570	980.73	988.40	11.601	3.0661	0.32615
4.0	1.0198	40.792	979.01	986.66	13.336	3.0588	0.32693
4.5	1.0233	46.048	977.25	984.89	15.106	3.0483	0.32805
5.0	1.0267	51.335	975.36	982.99	17.008	3.0183	0.33131
5.5	1.0302	56.661	973.54	981.15	18.848	3.0062	0.33265
6.0	1.0337	62.022	971.68	979.28	20.723	2.9929	0.33412
6.5	1.0373	67.424	969.88	977.46	22.540	2.9913	0.33430
7.0	1.0408	72.856	967.94	975.51	24.487	2.9753	0.33610
7.5	1.0443	78.322	965.98	973.53	26.468	2.9591	0.33794
8.0	1.0478	83.824	963.98	971.51	28.486	2.9426	0.33983
8.5	1.0514	89.369	962.03	969.55	30.446	2.9353	0.34068
9.0	1.0550	94.950	960.05	967.56	32.442	2.9268	0.34167
9.5	1.0586	100.57	958.03	965.52	34.478	2.9169	0.34283
10.0	1.0621	106.21	955.89	963.37	36.635	2.8991	0.34493
10.5	1.0658	111.91	953.89	961.35	38.651	2.8954	0.34538
11.0	1.0694	117.63	951.77	959.21	40.787	2.8840	0.34674
11.5	1.0730	123.40	949.60	957.03	42.974	2.8715	0.34825
12.0	1.0766	129.19	947.41	954.82	45.181	2.8594	0.34973
12.5	1.0803	135.04	945.26	952.65	47.348	2.8521	0.35062
13.0	1.0840	140.92	943.08	950.45	49.545	2.8443	0.35158
13.5	1.0877	146.84	940.86	948.22	51.782	2.8357	0.35264
14.0	1.0914	152.80	938.60	945.94	54.060	2.8265	0.35380
14.5	1.0951	158.79	936.31	943.63	56.368	2.8170	0.35498
15.0	1.0988	164.82	933.98	941.28	58.716	2.8071	0.35624
15.5	1.1026	170.90	931.70	938.99	61.014	2.8010	0.35702
16.0	1.1063	177.01	929.29	936.56	63.443	2.7901	0.35841
16.5	1.1101	183.17	926.93	934.18	65.821	2.7829	0.35934
17.0	1.1139	189.36	924.54	931.77	68.230	2.7753	0.36032
17.5	1.1177	195.60	922.10	929.31	70.689	2.7671	0.36140
18.0	1.1216	201.89	919.71	926.90	73.098	2.7619	0.36207
18.5	1.1254	208.20	917.20	924.37	75.628	2.7529	0.36325
19.0	1.1293	214.57	914.73	921.88	78.117	2.7468	0.36406
19.5	1.1332	220.97	912.23	919.36	80.636	2.7403	0.36492
20.0	1.1371	227.42	909.68	916.79	83.206	2.7332	0.36587
20.5	1.1410	233.90	907.10	914.19	85.806	2.7259	0.36685
21.0	1.1449	240.43	904.47	911.54	88.457	2.7180	0.36791
21.5	1.1489	247.01	901.89	908.94	91.057	2.7127	0.36864
22.0	1.1529	253.64	899.26	906.29	93.708	2.7067	0.36945
22.5	1.1569	260.30	896.60	903.61	96.389	2.7005	0.37030
23.0	1.1609	267.01	893.89	900.88	99.120	2.6938	0.37122
23.5	1.1649	273.75	891.15	898.12	101.88	2.6870	0.37216
24.0	1.1690	280.56	888.44	895.39	104.61	2.6820	0.37286
25.0	1.1772	294.30	882.90	889.80	110.20	2.6706	0.37445
26.0	1.1855	308.23	877.27	884.13	115.87	2.6601	0.37592
26.4	1.1888	313.84	874.96	881.80	118.20	2.6552	0.37663
26.8	1.1922	319.51	872.69	879.51	120.49	2.6518	0.37711

TABLE 1.—(*Continued.*)

TEMPERATURE = 60° CENT.

Volume of 1 Gramme of Distilled Water = 1.01705 c.c.

Percentage of salt solution.	Density, in grammes per cubic centimeter.	Concentration, in grammes per liter.	Weight of distilled water per liter of solution, in grammes.	Volume of distilled water per liter of solution, in cubic centimeters.	Apparent volume of salt per liter of solution, in cubic centimeters.
0.5	0.9869	4.9345	981.97	998.71	1.2920
1.0	0.9903	9.903	980.40	997.11	2.8872
1.5	0.9937	14.906	978.79	995.48	4.5176
2.0	0.9971	19.942	977.16	993.82	6.1815
2.5	1.0005	25.012	975.49	992.12	7.8799
3.0	1.0039	30.117	973.78	990.39	9.6140
3.5	1.0073	35.256	972.04	988.62	11.383
4.0	1.0107	40.428	970.27	986.82	13.185
4.5	1.0141	45.634	968.47	984.98	15.022
5.0	1.0175	50.875	966.62	983.11	16.894
5.5	1.0210	56.155	964.84	981.30	18.704
6.0	1.0244	61.464	962.94	979.35	20.646
6.5	1.0279	66.814	961.09	977.47	22.527
7.0	1.0314	72.198	959.20	975.56	24.444
7.5	1.0348	77.610	957.19	973.51	26.490
8.0	1.0383	83.064	955.24	971.52	28.477
8.5	1.0418	88.553	953.25	969.50	30.500
9.0	1.0453	94.077	951.22	967.44	32 559
9.5	1.0489	99.646	949.25	965.44	34.561
10.0	1.0524	105.24	947.16	963.31	36.691
10.5	1.0559	110.87	945.03	961.14	38.857
11.0	1.0595	116.54	942.96	959.04	40.963
11.5	1.0631	122.26	940.84	956.88	43.119
12.0	1.0666	127.99	938.61	954.61	45.387
12.5	1.0702	133.78	936.42	952.39	47.614
13.0	1.0738	139.59	934.21	950.14	49.862
13.5	1.0775	145.46	932.04	947.93	52.069
14.0	1.0811	151.35	929.75	945.60	54.398
14.5	1.0848	157.30	927.50	943.31	56.686
15.0	1.0884	163.26	925.14	940.91	59.086
15.5	1.0921	169.28	922.82	938.55	61.446
16.0	1.0958	175.33	920.47	936.16	63.836
16.5	1.0995	181.42	918.08	933.73	66.267
17.0	1.1032	187.54	915.66	931.27	68.728
17.5	1.1070	193.72	913.28	928.85	71.149
18.0	1.1107	199.93	910.77	926.30	73.701
18.5	1.1145	206.18	908.32	923.81	76.193
19.0	1.1183	212.48	905.82	921.26	78.736
19.5	1.1221	218.81	903.29	918.69	81.309
20.0	1.1259	225.18	900.72	916.08	83.923
20.5	1.1298	231.61	898.19	913.50	86.496
21.0	1.1336	238.06	895.54	910.81	89.191
21.5	1.1375	244.56	892.94	908.16	91.835
22.0	1.1414	251.11	890.29	905.47	94.531
22.5	1.1453	257.69	887.61	902.74	97.256
23.0	1.1492	264.32	884.88	899.97	100.03
23.5	1.1532	271.00	882.20	897.24	102.76
24.0	1.1572	277.73	879 47	894.46	105.54
25.0	1.1652	291.30	873.90	888.80	111.20
26.0	1.1733	305.06	868.24	883.04	116.96
26.4	1.1765	310.60	865.90	880.66	119.34
26.8	1.1798	316.19	863.61	878.33	121.67

TABLE 1.—(*Continued.*)

TEMPERATURE = 80° CENT.

Volume of 1 Gramme of Distilled Water = 1.02899 c.c.

Percentage of salt solution.	Density, in grammes per cubic centimeter.	Concentration, in grammes per liter.	Weight of distilled water per liter of solution, in grammes.	Volume of distilled water per liter of solution, in cubic centimeters.	Apparent volume of salt per liter of solution, in cubic centimeters.
0.5	0.9771	4.8855	972.21	......	......
1.0	0.9805	9.805	970.70	998.84	1.1646
1.5	0.9839	14.758	969.14	997.24	2.7626
2.0	0.9873	19.746	967.55	995.60	4.3966
2.5	0.9908	24.770	966.03	994.04	5.9648
3.0	0.9942	29.825	964.38	992.33	7.6678
3.5	0.9976	34.916	962.68	990.59	9.4078
4.0	1.0010	40.040	960.96	988.82	11.182
4.5	1.0044	45.198	959.20	987.01	12.991
5.0	1.0079	50.395	957.50	985.26	14.737
5.5	1.0113	55 622	955.68	983.38	16.617
6.0	1.0148	60.888	953.91	981.57	18.434
6.5	1.0182	66.183	952.02	979.62	20.384
7.0	1.0217	71.519	950.18	977.73	22.273
7.5	1.0251	76.832	948.22	975.71	24.293
8.0	1.0286	82.288	946.31	973.75	26.254
8.5	1.0320	87.720	944.28	971.65	28.345
9.0	1.0355	93.195	942.30	969.62	30.378
9.5	1.0390	98.705	940.30	967.55	32.446
10.0	1.0425	104.25	938.25	965.45	34.550
10.5	1.0460	109.83	936.17	963.31	36.690
11.0	1.0495	115.44	934.06	961.14	38.862
11.5	1.0530	121.10	931.90	958.92	41.084
12.0	1.0565	126.78	929.72	956.67	43.327
12.5	1.0601	132.51	927.59	954.48	45.519
13.0	1.0636	138.27	925.33	952.16	47.845
13.5	1.0671	144.06	923.04	949.80	50.201
14.0	1.0707	149.90	920.80	947.49	52.506
14.5	1.0743	155.77	918.53	945.16	54.842
15.0	1.0778	161.67	916.13	942.69	57.311
15.5	1.0814	167.62	913.78	940.27	59.730
16.0	1.0850	173.60	911 40	937.82	62.179
16.5	1.0886	179.62	908.98	935.33	64.669
17.0	1.0923	185.69	906.61	932.89	67.107
17.5	1.0959	191.78	904.12	930.33	69.670
18.0	1.0995	197.91	901.59	927.73	72.273
18.5	1.1032	204.09	899.11	925.18	74.825
19.0	1.1069	210.31	896.59	922.58	77.418
19.5	1.1105	216.55	893.95	919.87	80 134
20.0	1.1142	222.84	891.36	917.20	82.799
20.5	1.1179	229.17	888.73	914.49	85.506
21.0	1.1217	235.56	886.14	911.83	88.171
21.5	1.1254	241.96	883.44	909.05	90.949
22.0	1.1292	248.42	880.78	906.31	93.686
22.5	1.1329	254.90	878.00	903.45	96.547
23.0	1.1367	261.44	875.26	900.63	99.366
23.5	1.1405	268.02	872.48	897.77	102.23
24.0	1.1443	274.63	869.67	894.88	105.12
25.0	1.1520	288.00	864.00	889.05	110.95
26.0	1.1597	301.52	858.18	883.06	116.94
26.4	1.1628	306.98	855.82	880.63	119.37
26.8	1.1660	312.49	853.51	878.25	121.75

By determining the values of n for the various temperatures and then inserting these values in Equation (1) along with those of d for distilled water, there may be derived,

$$\left.\begin{aligned} d_0 &= 0.999868 + 0.772\,p_0 \\ d_{10} &= 0.999727 + 0.740\,p_{10} \\ d_{15} &= 0.999126 + 0.730\,p_{15} \\ d_{20} &= 0.998230 + 0.720\,p_{20} \\ d_{40} &= 0.992240 + 0.690\,p_{40} \end{aligned}\right\} \text{Limited to solutions of less than } \tfrac{1}{2}\% \text{ in salt} \;\ldots\ldots(2)$$

which, for percentages, may be rewritten as follows:

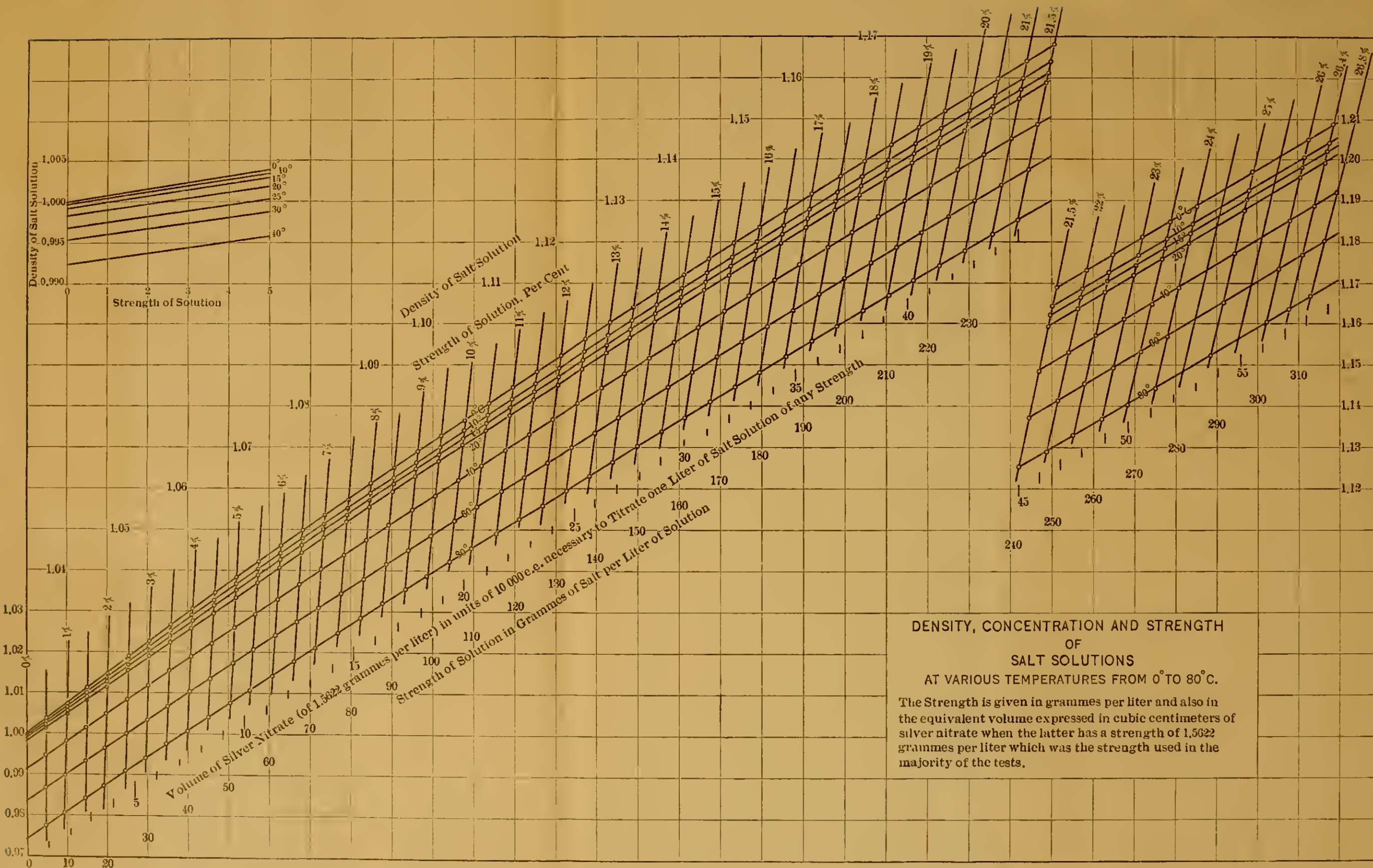
$$\left.\begin{aligned} 100\,p_0 &= 129.53d - 129.52 \\ 100\,p_{10} &= 135.14d - 135.10 \\ 100\,p_{15} &= 136.99d - 136.87 \\ 100\,p_{20} &= 138.89d - 138.64 \\ 100\,p_{40} &= 144.93d - 143.80 \end{aligned}\right\} \text{Limited to solutions of less than } \tfrac{1}{2}\% \text{ in salt} \;\ldots\ldots(3)$$

There may also be written:

$$\left.\begin{aligned} d_0 &= 0.499934 + \sqrt{0.249935 + 0.000772\,C} \\ d_{10} &= 0.499864 + \sqrt{0.249865 + 0.000740\,C} \\ d_{15} &= 0.499563 + \sqrt{0.249565 + 0.000730\,C} \\ d_{20} &= 0.499115 + \sqrt{0.249116 + 0.000720\,C} \\ d_{40} &= 0.496120 + \sqrt{0.246135 + 0.000690\,C} \end{aligned}\right\} \text{Limited to solutions of less than } 1\% \text{ in salt.} \;\ldots(4)$$

in which C is the concentration.

4.—For purposes of engineering and computation, a diagram plotted to a convenient scale will greatly facilitate determinations of the values of the quantities involved in Table 1 and the foregoing formulas without the necessity for calculations and interpolations. Plate XLVIII exhibits this plotting, the scale being sufficiently large to enable a reading of density to be made with great precision. This diagram, however, is not so accurate when it is required to ascertain the value of the concentrations of salt solutions in terms of the percentage. In order to ascertain concentrations with a sufficient degree of accuracy, Plates XLIX and L have been specially prepared. These diagrams give the relation between percentage and concentration with a high degree of precision when the temperature is between 10 and 30° cent., and may be used even beyond these limits. It is scarcely necessary to



DENSITY, CONCENTRATION AND STRENGTH
OF
SALT SOLUTIONS
AT VARIOUS TEMPERATURES FROM 0°TO 80°C.

The Strength is given in grammes per liter and also in the equivalent volume expressed in cubic centimeters of silver nitrate when the latter has a strength of 1.5622 grammes per liter which was the strength used in the majority of the tests.

make any further explanation, as the plotting will be sufficiently well understood.

5.—*Volumetric Shrinkage.*—When two salt solutions of different densities are mixed, the volume of the resultant mixture is less than the sum of the volumes of the two components before mixing. Table 1 and Plates XLVIII, XLIX, and L show this, as illustrated by the following example, the figures for which have been taken from Table 1:

Of a ½% solution, 100 liters are mixed with 1 liter of 25% solution at 20° cent. Find the percentage of the mixture, the density, and final volume, at the given temperature.

Solution:

	Kilogrammes of solution.	Kilogrammes of NaCl.
1 liter, 25% solution........	1.1879	0.29698
100 liters, ½% solution........	100.19	0.50095
Totals	101.3779	0.79793

$$\text{New percentage of the mixture} = \frac{0.79793}{101.3779} = 0.78708.$$

The density for this percentage is 1.0039.

Therefore the final volume of the mixture is $101.378 \div 1.0039 = 100.984$. Thus, the shrinkage of volume is $101 - 100.984 = 0.016$ liter.

This illustration shows that there is a shrinkage of volume, and that, for a ratio of mixture of 100 to 1, the relative shrinkage is very small.

Take another case, where the ratio of mixture is, say, only 1 to 1. Let 1 liter of 25% solution be mixed with 1 liter of distilled water at 20° cent. Find the percentage of the mixture at the given temperature.

Solution:

	Kilogrammes of solution.	Kilogrammes of salt.
1 liter, 25% solution.......	1.1879	0.29698
1 liter, distilled water......	0.9982	0.00000
Total weight	2.1861	0.29698

New percentage $= 0.29698 \div 2.1861 = 13.585$

Corresponding density $= 1.0975$ (Table 1).

Resulting volume $= 2.1861 \div 1.0975 = 1.99189$

Thus, the shrinkage of volume is $2 - 1.992 = 0.008$ liter.

Although this result is less than the former one, the relative error is much larger, being about 0.4% of the entire volume and 0.8% of the volume of the distilled water with which the strong solution was mixed. In this case the error is also 0.8% of the volume of the strong salt solution which is diluted with the distilled water.

These illustrations point to the fact that the shrinkage is not great, but may be sufficiently large to be taken into account when errors of less than 1% are to be computed.

6.—*Chemi-Hydrometry.*—As the method of measuring quantities of liquids by chemical procedure has not yet received a distinctive name, it may be appropriate to suggest one. The word "Chemi-hydrometry" seems to describe very exactly the nature of the process, but has three features which seem to be more or less unfavorable to its adoption.

The word seems to be too long to recommend itself. It is a compound of words derived from different languages, which is not good form. The last component has been used almost exclusively in reference to determinations of density and specific gravity, and is not so well known as relating to measurements of quantities of liquids.

However, until a better name is suggested, Chemi-hydrometry will be used by the writer as referring to the body of rules, formulas, and processes connected with the measurement of quantities of liquids by chemical processes.

7.—*Mixtures of Solutions.*—When two salt solutions of different percentages are mixed at a given temperature, the mixture, when at the same temperature, will not have a volume equal to the sum of the two volumes mixed, but will be more or less deficient. In consequence of this shrinkage of volume, it is necessary to develop a theory of mixtures before volumes can be determined by chemi-hydrometry with certainty.

By "ratio of mixture" is meant the ratio of the quantity of the weaker of two solutions to the quantity of the other with which it is mixed. These quantities may be measured in units of volume or units of weight, according to circumstances or convenience. The symbol for this ratio will be f for units of weight, and r for units of volume.

By "ratio of dilution" is meant the ratio of the quantity resulting from the mixture of two salt solutions of different percentages to the quantity having the higher percentage which has been mixed with

PLATE XLIX.
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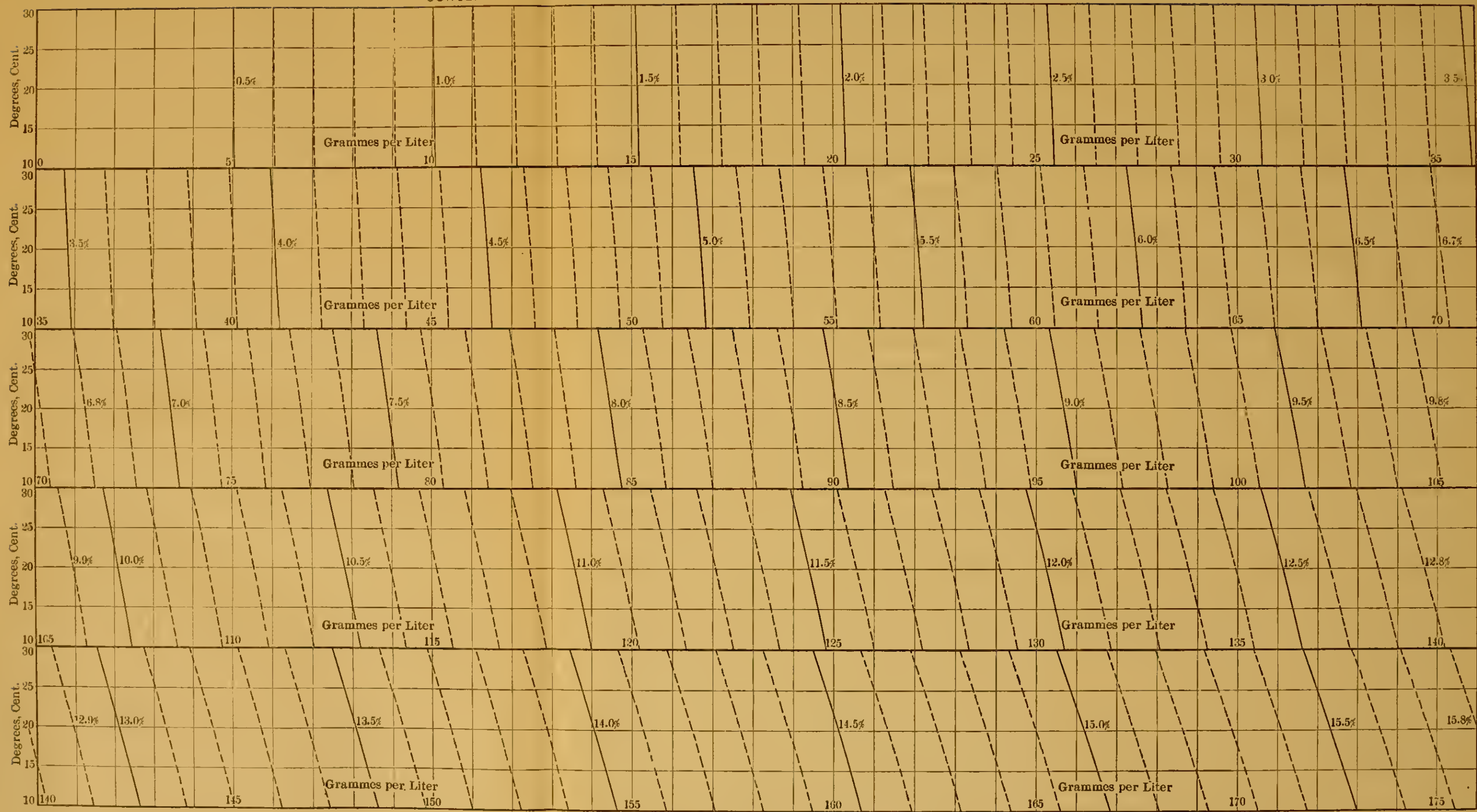
GROAT ON
CHEMI-HYDROMETRY.

CONCENTRATION AND STRENGTH OF SALT SOLUTIONS BETWEEN 10° AND 30° CENTIGRADE.



the other. The symbol for this ratio will be F for units of weight, and R for units of volume.

When a solution is referred to as "being of a certain percentage" it is meant that this percentage of the total weight of the solution is anhydrous chemical, or, in the case of salt solutions, simply dry salt.

By "strength of solution" will be meant either the percentage of the solution, or its equivalent, the ratio of the weight of salt in solution to the total weight of the solution. This latter ratio will be indicated by p. The percentage of a solution, therefore, is equal to 100 p.

By "concentration of a solution" is meant the weight of the chemical in solution per unit of volume; for example, the number of grammes of salt per liter of solution. The symbol, c, represents the concentration of a solution.

When quantities of solution are measured by weight, W and w will be used to represent these weights, and Q or q will represent volumes.

When quantities of silver nitrate, or other standard solution, are measured from a volumetric burette, in executing a titration, the symbol, t, will denote the volume measured, usually in cubic centimeters.

The quantity of a sample of solution to be titrated will be indicated by V in the case of measurement by volume and by S in the case of measurement by weight. Thus, $t \div V$ and $t \div S$ are volumes of silver nitrate consumed per unit quantity of sample, based on the titration, t.

The titration per unit quantity of sample will be denoted by v, so that equations of the form $v = t \div V$ and $v = t \div S$ will occur.

8.—Volume, Strength, Concentration, and Density of a Mixture of Several Known Salt Solutions.—Let the several known solutions be indicated by consecutive subscripts, as 1, 2, 3....n. Then the weight of the solution, x, is W_x, its strength is p_x, and the weight of the contained salt is $W_x\, p_x$. Therefore,

$$\text{Total weight of a mixture of } n \text{ solutions} = \Sigma\, W_x$$

$$\text{Total weight of salt in the } n \text{ solutions} = \Sigma\, W_x\, p_x$$

$$\text{Strength of the mixture of } n \text{ solutions} = p = \frac{\Sigma\, W_x\, p_x}{\Sigma\, W_x} \ldots\ldots (5)$$

The reader will not fail to observe that this formula is the same as that for a weighted average, which, of course, is the same as that for the average arm of several forces, or the "arm of gravity."

From the resulting value of p, it will be easy, with Plate XLVIII, to determine the density, d, of the mixture of the n solutions; and with Plates XLVIII, XLIX, or L, when great accuracy is required, the concentration, c, may be found.

Or, having found the density from Plate XLVIII, the concentration may be computed, thus:

$$\text{Concentration} = c = pd \qquad (6)$$

It will also be seen at a glance that

$$Q = \frac{\Sigma W_x}{d} \qquad (7)$$

9.—Mixture of Two Solutions.—For the ordinary purposes of chemi-hydrometry, mixtures of two solutions, one of which is strong while the other is comparatively weak, are of most interest. By omitting subscripts for the strong solution, using the subscript, 1, for the weaker component solution and 2 for the resultant mixture, the following special cases of Equations (5) and (7) may be written at once:

$$\left.\begin{aligned} p_2 &= \frac{wp + W_1 p_1}{w + W_1} \\ Q_2 &= \frac{w + W_1}{d_2} \end{aligned}\right\} \left\{\begin{array}{l} \text{Simple letters refer to strong salt solu-} \\ \text{tion. Subscript, 1, refers to 2d com-} \\ \text{ponent of mixture. Subscript, 2,} \\ \text{refers to resulting mixture.} \end{array}\right\} \qquad (8)$$

Equations (8) are rigorous, and may be used to determine the volume of a mixture of two solutions, in terms of the weights and strengths of the constituents, d_2 becoming known as soon as p_2 has been determined, as has been explained.

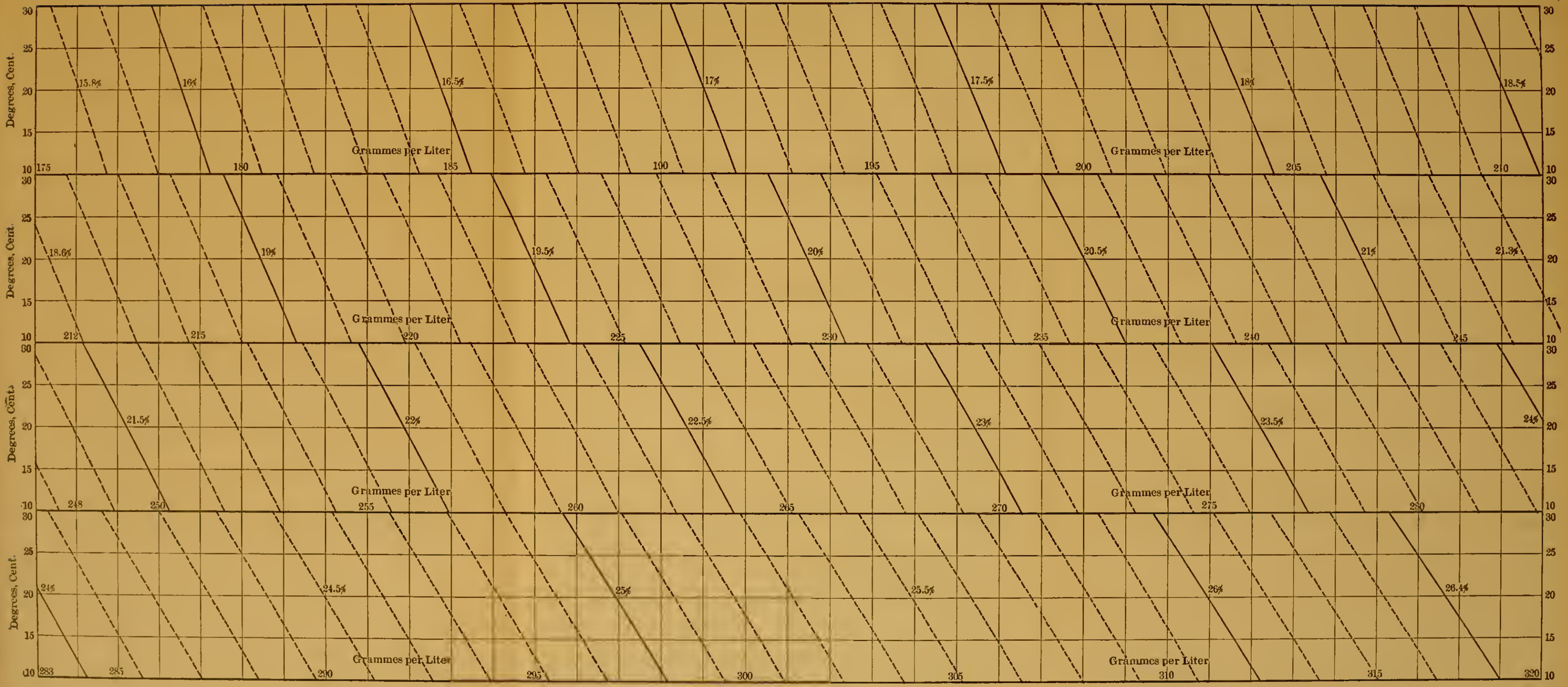
It is sometimes required to find the strength of a resultant solution in terms of the ratio of mixture, or of the ratio of dilution of two solutions, thus:

$$p_2 = \frac{p + f p_1}{1 + f} \quad f = \frac{W_1}{w} \qquad (9)$$

and

$$p_2 = \frac{(p - p_1) + F p_1}{F} \left\{\begin{aligned} F &= W_2 \div w \\ W_2 &= w + W_1 \end{aligned}\right\} \qquad (10)$$

CONCENTRATION AND STRENGTH OF SALT SOLUTIONS BETWEEN 10° AND 30° CENTIGRADE.



10.—Equations Based on Weights and Strengths Only.—It does not appear that any recognition has been given to the fact that, if the weights of the dosing solution and samples for titration are observed, it will be possible to compute the discharge of a turbine in units of weight, the only units really necessary for a determination of efficiency, without any temperature corrections. The titrations of the samples, of course, are measured in units of volume of silver nitrate, the same as in all cases.

It is very clear, therefore, that much of the labor of computation may be saved by using units of weight in determining quantities of solution and sizes of samples, rather than units of volume. This, however, makes somewhat more trouble in calibrating the chemical tank, as the weight of solution consumed depends on the density, where it is the volume taken from the tank that is measured. On the other hand, this is not so serious a matter as the calculations necessary to ascertain the corrections to be applied to titrations, volumes, and densities.

It follows from this that the ideal method of measuring the dosing solution is by weight, and not by volume. Where this is possible, the method of weights should be adopted unqualifiedly. The importance of this, in the case considered herein, was not realized in time to arrange the testing equipment for the method of weights. However, as the theory for this method is simple, and promises much for the future, it will be given first.

It should be observed, first, that the chemical method determines quantities, that is, weights or volumes of water, or other liquid. Consequently, it is applicable for finding the capacities of tanks and reservoirs, as the volume of such a receiver is equal to the volume of liquid which fills it. Indeed, the capacity of a reservoir, up to any given elevation, can be found immediately by simply ascertaining the quantity of liquid it contains when filled up to that elevation.

This can be done in two ways, one or the other of which may be found the more convenient. The tank may be first filled to the given elevation with the normal water from a water supply, after which an appropriate quantity of salt solution is mixed with the normal water. Or, the tank being filled nearly to the given elevation with normal water, salt solution may be added and mixed until the mixture

rises to that elevation. In the first case it is required to find the volume of the normal water before introducing salt, and in the second case it is required to find the volume of the mixture.

In both cases the necessary data are the quantity of salt solution mixed with the normal water and the respective strengths of the salt solution, the normal water, and the mixture. The strengths are determined by chemical analyses, usually by titrations of the samples.

The first of these two cases corresponds to determining the quantity of water passing through a turbine when the salt is injected into the tail-water discharged from the turbine, and samples of the mixture are taken at a point still farther down stream. The second case corresponds to a determination of the quantity of mixture discharged from a turbine when the salt is injected into the head-water, and the mixture is sampled either in the head-race, before it reaches the turbine, or in the tail-race, after passing through the turbine. The difficulties of sampling are discussed in Section 55 *et seq.* See also Sections 58, 59, and 84.

11.—It will facilitate comprehension of the equations to remember that letters without subscripts relate to salt solutions, Subscript 1 relates to normal water, and Subscript 2 relates to the resultant mixture. Thus, w is a quantity of salt solution considered either as a whole, in the case of calibrating a tank, or as a rate of quantity; that is, a quantity per unit of time, as in the case of a turbine test. It may be well, also, to remember that the normal water may contain salt or other chemical initially, thus complicating the equations to a certain extent. Therefore:

Weight of salt in normal water.......... $= W_1 \, p_1$
" " " " salt solution .... $= w \, p$
" " " " resultant mixture ...... $= W_2 \, p_2$

From which

$$W_1 \, p_1 + w \, p = W_2 \, p_2 \qquad (11)$$

But

$$W_2 = W_1 + w$$

$$W_1 \div w = f$$

$$W_2 \div w = F$$

Therefore,

$$\left.\begin{aligned} f \; + 1 &= F, \\ f \, p_1 + p &= F \, p_2, \end{aligned}\right\} \qquad (12)$$

Thus,

$$\left.\begin{aligned} f &= \frac{Fp_2 - p}{p_1} \\ F &= \frac{fp_1 + p}{p_2} \end{aligned}\right\} \dots\dots\dots\dots\dots\dots (13)$$

Again, by eliminating first F, and then f, from Equations (12),

and

$$\left.\begin{aligned} f &= \frac{p - p_2}{p_2 - p_1} \\ F &= \frac{p - p_1}{p_2 - p_1} \end{aligned}\right\} \dots\dots\dots\dots\dots\dots (14)$$

Equations (14) give the ratio of mixture and the ratio of dilution in terms of the strengths of the three solutions involved. With these ratios may be calculated at once the volumes of the normal head-water and resultant mixture, thus

and

$$\left.\begin{aligned} W_1 &= wf \\ W_2 &= wF \end{aligned}\right\} \dots\dots\dots\dots\dots\dots (15)$$

Now, if the end points of the titrations are in error by the same relative amount, we should have

$$\left.\begin{aligned} p &= Ev \\ p_1 &= Ev_1 \\ p_2 &= Ev_2 \end{aligned}\right\} \dots\dots\dots\dots\dots\dots (16)$$

as each unit of volume of silver nitrate would correspond to a definite quantity of salt. Consequently, we may write at once, noting that E is a common factor,

and

$$\left.\begin{aligned} W_1 &= w\,\frac{v - v_2}{v_2 - v_1} \\ W_2 &= w\,\frac{v - v_1}{v_2 - v_1} \end{aligned}\right\} \dots\dots\dots\dots\dots\dots (17)$$

v being the titrations per kilogramme, or other unit of weight, reduced from actual titrations of samples in the laboratory.

It must be carefully observed that this substitution of v for p throughout an equation of chemi-hydrometry is permissible only where the error introduced is either inappreciable or entirely removed by correcting the titrations when seriously affected. The real nature of the errors committed by such substitutions will become more apparent with further study of the subject.

12.—Special Laboratory Procedure and Special Dilutions.—For the present it will suffice to state that the writer eliminates this error by resort to a special laboratory process, the principle involved being as follows:

To have two or more titrations in error by the same amount, and therefore by the same relative amount, the samples must be of exactly the same size and just alike in all respects; the standard solution and the indicator must each have the same strength in the two or more cases; and, in fact, the samples and titrations in the several cases must be just alike in all other respects as regards the conditions under which the operations take place.

The truth of this proposition, being axiomatic, needs no comment. The application of the principle is effected by making a special dilution in the laboratory which is to have the same ratios of mixture and dilution as has the mixture of salt solution and normal feed-water in the actual test. Of course, the same salt solution and normal feed-water must be used in the laboratory as on the test.

This procedure cannot be accomplished without advance knowledge of the actual ratio of dilution. In any case, it will always be possible to have a more or less approximate value for this ratio, and the approximate value may be used in making up preliminary special dilutions, from which a close approximation to the true ratio may be computed by the methods immediately to follow, thus leading to a second approximation, which, in most cases, will be sufficiently exact. In fact, it will be found in most actual tests that the true value of the ratio will be known in advance with sufficient closeness to enable the making of satisfactory special dilutions.

It will seldom chance to be the case, however, that the ratios of mixture and dilution actually used in the laboratory will prove to be equal to the true ratios corresponding. It is necessary, therefore, to distinguish carefully between the actual ratios, samples, and titrations and the special ratios, samples, and titrations of the laboratory dilutions. A prime, therefore, will be affixed to the letters and symbols pertaining to the special dilutions; thus, r', R', and w' are, respectively, the ratio of mixture, ratio of dilution, and weight of salt solution used in making up a special dilution.

13.—Assume, then, that a test has actually been made, that samples of percentages, p and p_2, have been taken, and that w has been

measured. Further, that special dilutions have been prepared, and that the quantities, r', R', w', p', and p_2', have also been observed. There may then be written, from Equations 14, Section 11, the following relations:

$$\left.\begin{aligned} \frac{F}{f} &= \frac{p - p_1}{p - p_2}, \text{ for the actual test,} \\ \text{and} \quad \frac{f'}{F'} &= \frac{p - p_2'}{p - p_1}, \text{ for the special dilutions.} \end{aligned}\right\} \ldots\ldots\ldots (18)$$

It may be helpful here to remark that p and p_1 are the same in both preceding equations, as the same salt solution and the same normal feed-water have been used in the laboratory and on the actual test. Further, the unknown, p_1, may now be eliminated from Equations (18), thus:

$$\frac{Ff'}{fF'} = \frac{p - p_2'}{p - p_2} = 1 + \frac{p_2 - p_2'}{p - p_2} \ldots\ldots\ldots\ldots (19)$$

Now put

$$\left.\begin{aligned} x &= \frac{p_2 - p_2'}{p - p_2} \\ \text{and} \quad y &= \frac{p_2 - p_2'}{p - p_2'} \end{aligned}\right\} \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

from which

$$\frac{x}{y} = \frac{1}{1 - y} = 1 + x \ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Therefore,

$$\frac{Ff'}{fF'} = 1 + x, \text{ or}$$

$$1 + \frac{1}{f} = \left(1 + \frac{1}{f'}\right)(1 + x) = 1 + \frac{1}{f'} + \frac{F'\,x}{f'} \ldots\ldots (22)$$

as $F = f + 1$, and $F' = f' + 1$, by Equations (12), Section 11.

From Equation (22) may be written at once

$$f = \frac{f'}{1 + F'\,x}, \text{ and } \left(x = \frac{p_2 - p_2'}{p - p_2}\right) \ldots\ldots\ldots\ldots (23)$$

Again,

$$\frac{fF'}{Ff'} = \frac{1}{1 + x} = 1 - y \ldots\ldots\ldots\ldots\ldots\ldots (24)$$

or

$$1 - \frac{1}{F} = \left(1 - \frac{1}{F'}\right)(1 - y) \ldots\ldots\ldots\ldots\ldots (25)$$

which by analogy gives at once

$$F = \frac{F'}{1 + f'\,y}, \text{ and } \left(y = \frac{p_2 - p_2'}{p - p_2'}\right) \ldots\ldots\ldots\ldots (26)$$

If it is required to have equations in terms of either the rate of mixture or rate of dilution, they may be written out as follows, by simply putting $F - 1$ for f, or $f + 1$ for F, thus:

Method of special dilutions. Equations based on weights of samples.

$$\left\{\begin{aligned} f &= \frac{f'}{1 + (f' + 1)\,x} \\ f &= \frac{F' - 1}{1 + F'\,x} \\ F &= \frac{f' + 1}{1 + f'\,y} \\ F &= \frac{F'}{1 + (F' - 1)\,y} \end{aligned}\right\} \left\{\begin{aligned} &\left(x = \frac{p_2 - p_2'}{p - p_2}\right) \\ &\left(y = \frac{p_2 - p_2'}{p - p_2'}\right) \end{aligned}\right\} \quad (27)$$

14.—It will not always be advisable to eliminate p_1 in this manner. Frequently, it will be advisable to determine p_1 by Equations (14), from the known ratio of mixture, or ratio of dilution, used in the preparation of the special dilutions, and then determine F, or f, by a second application of Equations (14), with the resulting value of p_1 introduced.

An advantage secured by this procedure arises when the normal feed-water for the turbine runs constant in salt content. In this case the value of p_1, reduced to standard conditions, should be the same in all the tests, if care has been taken to keep the silver nitrate at the same standard strength. An error in the laboratory process is then made apparent by a discrepancy among the corrected values of p_1.

The laboratory procedure is exactly the same as before, the only difference being in the method of calculation, which is sufficiently indicated by the following equations, derived from Equations (13) or (14).

By the method of "Special Dilutions."

$$\left\{\begin{aligned} p_1 &= \frac{F'\,p_2' - p}{f'} \quad \text{(a check on uniformity of normal head-water)} \\ f &= \frac{p - p_2}{p_2 - p_1} \\ \text{or, } F &= \frac{p - p_1}{p_2 - p_1} \end{aligned}\right\} \ldots (28)$$

15.—Object of the Special Dilutions.—The influence of the special dilutions toward eliminating the errors of titration may now be more fully explained.

An inspection of Equations (14), Section 11, will show at a glance that, if all the strengths of the solutions as determined by the titrations are in error by the same relative amount, the ratios of mixture and dilution, and therefore the quantities of liquid to be measured, will, nevertheless, be correctly determined by the formulas.

If the percentages, p, p_1, p_2, however, are not in error by the same relative discrepancy, then an error of greater or less moment will be introduced in the calculations, and, as a consequence, they are based on fallacious data. It should be observed here that p is relatively much larger than p_2 and p_1, and that p_2 may be any number of times larger than p_1. Thus, in the case of the tests to be discussed, the salt solution used was near the saturation point, but the normal canal water carried about 15 mg. of salt per liter. The quantity of salt injected into the head-water was from 3 to 4 times as much as the water contained initially.

The corresponding values of v, v_2, and v_1, therefore, were about 540 000, 110, and 25 c.c. per liter of the respective solutions, when the silver nitrate was of a concentration of about 1.56 grammes per liter of distilled water. The respective values of 100 p, 100 p_2 and 100 p_1, therefore, were about 24.5%, 0.00592%, and 0.00134%, and c, c_2, and c_1, were about 290, 0.0591, and 0.0134 grammes per liter of the corresponding sample.

Table 2 facilitates a comparison of these figures.

TABLE 2.—Comparison of Salt and Silver Nitrate Equivalents, and Strengths and Densities of Salt Solution, Head-Water, and Tail-Water Samples Involved in the Tests.

$AgNO_3$ solution contains about 1.56 grammes per liter of distilled water.

Quantities compared.	Salt solution.	Tail-water.	Head-water.
Cubic centimeters of silver nitrate per liter of sample..	540 000	110	25
Ditto per kilogramme of sample..........................	456 200	110.2	25
Percentage strength of solution (100p)...................	24.50	0.00592	0.001342
Densities..	1.1837	0.9984	0.9984
Concentration, in grammes per liter......................	290	0.0591	0.0134

Thus, by Equations (14), Section 11, the ratio of dilution by weight will be:

$$F = \frac{24.50 - 0.00134}{0.00592 - 0.00134} = \frac{24.50}{0.00458} = 5\,349.3 \ldots\ldots (29)$$

or, for every pound of salt solution injected into the head-race there were, as an average, 5 349 lb. of mixture flowing through the turbine.

Now the percentages, 100 $p_1 = 0.00134$, and 100 $p_2 = 0.00592$, are both in error relative to 100 $p = 24.50$, as the titrations were not of samples which were alike in all respects. For example, the titration for p is made by first diluting 10 c.c.* of salt solution to 1 liter and then placing 10 c.c. of this dilution in a casserole for the actual titration, which, therefore, would be about 54 c.c. of silver nitrate, the ten-thousandth part of what it would be for a whole liter of salt solution. On the other hand, the titrations of the normal head-water and of the tail-water are made by evaporating 500 c.c. of sample to about 10 c.c. in a casserole before making the titrations. Thus, these samples have been reduced by evaporation, whereas the salt solution sample has been increased in size by diluting with distilled water. Moreover, the concentration of the tail-water sample, and consequently its titration, are each about 4½ times what they are for the normal head-water, thus rendering p_2 and p_1 relatively inconsistent with each other as well as with p.

Under the conditions cited, it can be shown that the titration of normal head-water should be decreased about 0.14 c.c. for each ½-liter sample, a little more than 1%; and the titration of the tail-water, the ½ liter evaporation titrating so nearly equal to the titration of the salt solution, namely, about 55 c.c., needs scarcely any correction. Therefore, a more correct value for the ratio of dilution will be

$$F = \frac{24.5}{0.00592 - 0.001325} = 5\,332 \ldots\ldots\ldots\ldots\ldots (30)$$

which is seen to differ from the uncorrected ratio by about ⅓ of 1%, which, relatively, was too large an error to pass without notice in the turbine tests. The object of the special dilutions is to eliminate this error without the necessity for determining the corrections to be applied to the titrations.

16.—Function of the Special Dilutions.—The operation of the special dilutions to eliminate the errors of titration may be illustrated by an application of any of Equations (27) to the problem of the preceding paragraph.

* The method of weights was not used in the tests in question. Hence volumes instead of weights are given here.

First, let it be observed that the errors of titration can affect only x and y of the equations, because the results of the titrations enter only these quantities, and f' or F' are determined independently by the relative weights of the samples. Consequently, the effect of an error can be discovered by ascertaining the effect on x, or y, and then introducing the erroneous value of x, or y, in the corresponding equation.

Let S, S_2, and S'_2 be the weights of the samples at the times they are actually titrated. Let ρ_2, and ρ'_2 be the ratios of evaporation of the tail-water and special dilutions, that is, the ratio of the measured weight of a sample to its weight when ready for titration after the evaporation. In order to have the titrations as nearly alike as possible, the dilution of the salt solution and evaporations of the tail-water and special dilutions must be proportioned so that the actual samples titrated will be as nearly alike as possible. This will mean that the actual weights titrated and the strengths of these samples must be the same, and, therefore, their volumes must be the same. Thus, if ρ is the ratio of dilution of the salt solution with distilled water, c_s the concentration of the silver nitrate, and e the actual equivalent weight of salt per unit weight of silver nitrate consumed in titration, we must have,

$$\left.\begin{aligned} p &= v\, c_s\, e = \rho \frac{t}{S} c_s\, e \\ p_2 &= v_2\, c_s\, e_2 = \frac{t_2}{\rho_2\, S_2} c_s\, e_2 \\ p_2' &= v_2'\, c_s\, e_2' = \frac{t_2'}{\rho_2'\, S_2'} c_s\, e_2' \end{aligned}\right\} \qquad (31)$$

If the laboratory procedure is properly arranged, R' will be the same as R, so that the titrations for t_2 and t'_2 will be exactly alike, and, therefore, $t_2 = t'_2$, $\rho_2 = \rho'_2$, $S_2 = S'_2$, and $e_2 = e'_2$, but e will not necessarily be equal to either e_2 or e'_2, owing to the dilution of the salt solution with distilled water instead of normal head-water, and to other obvious differences among the conditions under which the operations on the salt solution and the other two kinds of samples take place.

However, we may write

$$\left.\begin{aligned} S &= S_2 = S_2' \\ e_2 &= e_2' \\ \text{and} \quad \rho_2 &= \rho_2' \end{aligned}\right\} \qquad (32)$$

With Equations (31) and (32) we may substitute in the values of x and y to obtain

$$\left.\begin{aligned} x &= \frac{p_2 - p_2'}{p - p_2} = \frac{v_2 - v_2'}{v - v_2} = \frac{t_2 - t_2'}{T - t_2} \\ y &= \frac{p_2 - p_2'}{p - p_2'} = \frac{v_2 - v_2'}{v - v_2'} = \frac{t_2 - t_2'}{T - t_2'} \end{aligned}\right\} \ldots\ldots\ldots\ldots(33)$$

where
$$T = \rho\, \rho_2 \frac{e}{e_2} t$$

which shows that, when $t_2 = t'_2$, the samples having been made exactly alike, $x = 0$ and $y = 0$, whether there is any error in t, that is, in T, or the contrary. It is clear, therefore, that:

When the ratio of dilution for the special dilutions is the same as that in the actual test, Equations (27) are rigidly correct and independent of any error in the titration of the salt solution.

We may go further, and say:

When the difference between t_2 and t'_2 is sufficiently small, the error in t will have no appreciable effect on the values of the ratios computed by Equations (27), and it is also apparent that any errors due to the inequality of t_2 and t'_2 will be negligible under similar conditions.

Now, to apply Equations (27) to the problem of Section 15, we must suppose a special dilution made up by taking, say, 10 grammes of salt solution, diluting this to 1 000 grammes with normal head-water and then diluting 10 grammes of the resultant dilution to, say, 600 grammes, so as to produce a dilution of 1 to 6 000, or $R' = 6\,000$. Breaking up a dilution in this way keeps down the size of the quantities treated in the laboratory.*

Under this treatment, a sample of 500 grammes of the dilution would, after evaporation to about 10 grammes, titrate to 50.335 grammes of silver nitrate. This evaporation to 10 grammes requires that the sample of dilute salt solution should also be of 10 grammes weight, though a dilution of 10 grammes to 1 kg. would cause the titration to be one ten-thousandth of the titration for 1 kg., as shown in Table 2, Section 15, which is 45.62 c.c., or $t = 45.62$. All this

* In making successive dilutions, however, it is always better to make the partial dilutions equal, wherever practicable, this being on the side of economy and accuracy. Thus, if the two flasks had equal capacities of 750 c.c., the ratio would have been $75^2 = 5\,625$, instead of 6 000. The latter ratio is used merely for the purpose of a particular illustration.

requires that $\rho = 100$ and $\rho_2 = 50$, so that $\rho\,\rho_2 = 5\,000$. If we neglect the error in $e \div e_2$, calling this ratio unity, we may write

$$T = \rho\,\rho_2\,t\,\frac{e}{e_2} = 5\,000 \times 45.62 = 228\,100 \ldots\ldots\ldots\ldots (34)$$

Simultaneously with the evaporations of the special dilution, there would be evaporated a number of tail-water samples which, in accord with the problem of Section 15, titrate to 55.10 c.c. of silver nitrate, or $t_2 = 55.10$.

The value of y then becomes

$$y = \frac{55.10 - 50.335}{228\,100 - 50.335} = 0.00002089 \ldots\ldots\ldots\ldots (35)$$

Now, it can be shown that the errors due to titration, under the circumstances of the problem, would not exceed 0.01 c.c. in the titration of t'_2, and not more than 0.03 c.c. in the titration of t, in order to leave the same relative error in all three titrations. See Section 43.

Indeed, these assumed errors are certainly large rather than small, as will appear in the study of errors, Section 42, *et seq.* Hence, affecting the titrations, t, t'_2, with these errors, which are to be subtracted in both cases, we shall have for the new value of y, observing that now $e \div e_2$ is properly made equal to unity,

$$\text{corrected } y = \frac{55.10 - 50.325}{227\,950 - 50.0} = 0.00002095 \ldots\ldots\ldots (36)$$

By the last of Equations (27) the ratio of dilution in the tail-race will be, for Equations (35) and (36), respectively,

$$\left.\begin{aligned} F_{35} &= \frac{6\,000}{1 + 5\,999\,y_{35}} = 5\,332 \\ F_{36} &= \frac{6\,000}{1 + 5\,999\,y_{36}} = 5\,330 \end{aligned}\right\} \ldots\ldots\ldots\ldots\ldots (37)$$

Thus, by neglecting the errors of titration, when $t = 45.62$, $t_2 = 55.10$, $t'_2 = 50.325$, the method of special dilutions gives the discharge correct to 2 in 5 332, or with an error of less than 0.04%, because, by applying the corrections as in Equation (36), the true ratio of dilution is seen to be 5 330.

Now, in all actual cases, t, t_2, and t'_2 can, without much trouble, be brought into much closer agreement than has been here supposed, and by a second treatment they may even be made equal if desired. Hence, it is clear that the laboratory work can be made to yield results which

are precise to one or two hundredths of 1%, in so far as systematic errors are concerned. As to accidental errors, there is no difficulty in checking the means of two sets of six or eight titrations to within 0.04 or 0.05%, so that a precision of $\frac{1}{20}$ of 1% is not difficult to realize.

If Equation (30) and the last of Equations (37) were actual cases, instead of mere illustrative problems, it would be wise to consider the latter as the more exact, or as having the greater weight, because errors in t, that is, in the numerator of Equation (30) and in the denominators of x and y, have much less effect on Equations (27) than on Equations (14).

Although the problems of this and the preceding section have been given as merely illustrative, the suppositions are based on the figures of Tables 2, 4, 5, and 6, which are derived from the results of actual tests. The errors thus computed, therefore, are very close estimates of any actual errors that would occur under similar circumstances.

17.—Ratio of Evaporation Nominal Only.—The ratio of evaporation has been defined and introduced into Equations (31) and (33). It should be observed that this is merely a nominal ratio of evaporation, as the evaporation is not a measured quantity. Its real significance in the equations may be better understood as being the ratio of the weight of a sample, as measured for evaporation, to the weight of a sample of dilute salt solution, as measured for titration. Thus, S is the weight of a dilute salt solution sample, say 10 grammes, and $\rho_2 S_2$ is the true weight of a tail-water sample, about 500 grammes, S_2 being equal to S for samples of equal volume. Thus $\rho_2 = 50$ in that case.

The evaporations will be closely enough equal if they are all made as nearly equal to the volume of a dilute salt solution sample as the eye can estimate by actual comparison when both samples are in exactly equal casseroles. A dilute salt solution sample can be used as a "check" for the purpose of this comparison.

18.—Volumetric Equations Involving Densities and Strengths.—Equations (27) contemplate the measurement of samples by weight, and determine the weight of water passing through the turbine during a test. It is more common in chemistry to measure quantities of liquid samples by volume. This will add somewhat to the complexity of the

equations, as has been explained. However, the equations will prove convenient for many purposes, and are therefore set forth, as follows:

According to the definitions, Section 7, we must have

$$\left.\begin{aligned} f &= \frac{W_1}{w} = \frac{Q_1\,d_1}{q\,d} = r\,\frac{d_1}{d} \\ F &= \frac{W_2}{w} = \frac{Q_2\,d_2}{q\,d} = R\,\frac{d_2}{d} \\ f' &= \frac{W'_1}{w'} = \frac{Q'_1\,d_1}{q'\,d} = r'\,\frac{d_1}{d} \\ F' &= \frac{W'_2}{w'} = \frac{Q'_2\,d'_2}{q'\,d} = R'\,\frac{d'_2}{d} \end{aligned}\right\}\quad\ldots\ldots\ldots\ldots(38)$$

Hence, in order to introduce volumes and densities in Equations (27), Section 14,* it will be necessary merely to substitute these values of the weight ratios in the proper places and thus obtain

$$\left.\begin{aligned} r &= \frac{r'}{1+\left(r'\,\frac{d_1}{d}+1\right)x} & x &= \frac{p_2-p'_2}{p-p_2} \\ r &= \frac{R'\,\frac{d'_2}{d_1}-\frac{d}{d_1}}{1+R'\,\frac{d'_2}{d}\,x} & & \\ R &= \frac{\left(r'\,\frac{d_1}{d}+1\right)}{1+r'\,\frac{d_1}{d}\,y} & & \\ R\dagger &= \frac{R'\,\frac{d'_2}{d_2}}{1+\left(R'\,\frac{d'_2}{d}-1\right)y} & y &= \frac{p_2-p'_2}{p-p'_2} \end{aligned}\right\}\quad\ldots(39)$$

By the method of special dilutions, the equations involving densities and strengths.

19.—Volumetric Equations Involving Concentrations and Strengths.—These equations may be deduced by substituting $c \div p$ for d, etc., in Equations (39), but the following method is easier and serves better

---

* These substitutions may be made in other equations. For example, in Equations (14), Section 11, to obtain equations involving r and R.

† It is remarked that this last equation is really the only one which is entirely free from the properties of the normal head-water. Also notice that concentrations, densities, and percentages are connected by Table 1, Equations 2, 3, 4, and Plates XLVIII, XLIX, and L, so that, any one being given, the other two may be found. Thus these equations express the volumetric ratios in terms of either density or percentage when taken in conjunction with the tables, equations, or diagrams.

to familiarize one with the relations existing among the quantities involved. It is first to be remembered that Equations (39) were established by the use of Equations (14), Section 11, Equations (20-21), Section 13, Equations (27), Section 14, and Equations (38), Section 18. Further, let

W_{s_1} = the total weight of salt in the head-water;
W_{s_2} = " " " " " " " tail-water;
w_s = " " " " " " " salt solution;
$f_s = W_{s_1} \div w_s$;
$F_s = W_{s_2} \div w_s$;
$\boldsymbol{P}$ = ratio of total weight of salt solution to weight of salt content;
$\boldsymbol{P}_1$ = ratio of total weight of head-water to weight of salt content;
$\boldsymbol{P}_2$ = ratio of total weight of tail-water to weight of salt content.

Thus

$$\boldsymbol{P} = 1 \div p, \text{ etc.}$$

Then

$$W_{s1} \boldsymbol{P}_1 + w_s \boldsymbol{P} = W_{s2} \boldsymbol{P}_2$$
$$W_{s1} + w_s = W_{s2}$$

or

$$f_s \boldsymbol{P}_1 + \boldsymbol{P} = F_s \boldsymbol{P}_2$$

and

$$f_s + 1 = F_s$$

from which

$$\left.\begin{aligned} f_s &= \frac{\boldsymbol{P} - \boldsymbol{P}_2}{\boldsymbol{P}_2 - \boldsymbol{P}_1} \\ F_s &= \frac{\boldsymbol{P} - \boldsymbol{P}_1}{\boldsymbol{P}_2 - \boldsymbol{P}_1} \end{aligned} \text{ (See Equations (14))} \right\} \cdots\cdots\cdots\cdots (40)$$

Hence, if we put

$$\left.\begin{aligned} X &= \frac{\boldsymbol{P}_2 - \boldsymbol{P}'_2}{\boldsymbol{P} - \boldsymbol{P}_2} \\ Y &= \frac{\boldsymbol{P}_2 - \boldsymbol{P}'_2}{\boldsymbol{P} - \boldsymbol{P}'_2} \end{aligned} \text{ (See Equations (20))} \right\} \cdots\cdots\cdots\cdots (41)$$

Also, similar to Equations (38),

$$\left.\begin{aligned} f_s &= \frac{W_{s1}}{w_s} = \frac{Q_1 c_1}{q\ c} = r \frac{c_1}{c} \\ F_s &= \frac{W_{s2}}{w} = \frac{Q_2 c_2}{q\ c} = R \frac{c_2}{c} \end{aligned} \text{ (See Equations (38))} \right\} \cdots\cdots (42)$$

Therefore, by analogy, we are able to write out at once, from a mere inspection of Equations (27),

$$\left.\begin{aligned}
f_s &= \frac{f'_s}{1 + (f'_s + 1)\,X} \\
f_s &= \frac{F'_s - 1}{1 + F'_s\,X} \\
F_s &= \frac{f'_s + 1}{1 + f'_s\,Y} \\
F_s &= \frac{F'_s}{1 + (F'_s - 1)\,Y}
\end{aligned}\quad \text{(See Equations (27))}\right\} \ldots\ldots\ldots (43)$$

By further analogy, from Equations (39), substituting *c* for *d*, etc.,

$$\left.\begin{aligned}
r &= \frac{r'}{1 + \left(r'\frac{c_1}{c} + 1\right)\frac{p}{p'_2}\,x} & X &= \frac{p}{p'_2}\,x \\
r &= \frac{R'\frac{c'_2}{c_1} - \frac{c}{c_1}}{1 + R'\frac{c'_2}{c}\,\frac{p}{p'_2}\,x} & &\text{By the method of special dilutions, the equations involving concentrations and strengths.} \\
& & &\text{(See Equations (39))} \\
R &= \frac{r'\frac{c_1}{c} + 1}{1 + r'\frac{c_1}{c}\,\frac{p}{p_2}\,y} & Y &= \frac{p}{p_2}\,y \\
R &= \frac{R'\frac{c'_2}{c_2}}{1 + \left(R'\frac{c'_2}{c} - 1\right)\frac{p}{p_2}\,y}
\end{aligned}\right\} \ldots (44)$$

20.—It must not be overlooked that in some cases it may be of more advantage to compute the strength, or other property, of the normal head-water, from the special dilutions than to eliminate it entirely from the equations. Thus, for example, by substituting in Equations (28) from Equations (38), there may be obtained

$$\left.\begin{aligned}
p_1 &= \frac{R'\frac{d'_2}{d}\,p'_2 - p}{r'\frac{d_1}{d}} & &\text{(See Equations (28))} \\
\text{or}\qquad p_1 &= \frac{R'\frac{d'_2}{d}\,p'_2 - p}{R'\frac{d'_2}{d} - 1} & &\text{(By the method of special dilutions, values of } p_1 \text{ to check constancy of salt content in head-water.)}
\end{aligned}\right\} \ldots\ldots (45)$$

$$\left.\begin{aligned} r &= \frac{d}{d_1}\,\frac{p-p_2}{p_2-p_1} \\ R &= \frac{d}{d_2}\,\frac{p-p_1}{p_2-p_1} \end{aligned}\right. \qquad \left\{\begin{aligned} f &= r\,\frac{d_1}{d} \\ F &= r\,\frac{d_2}{d} \end{aligned}\right\} \ldots\ldots\ldots\ldots (46)$$

which, by giving the value of p_1, d_1, etc., may serve as a check on the constancy of head-water concentration, density, etc.

Observe that the first of Equations (46) may be used to find the volume of a quantity of solution with which a known volume of standard solution has been mixed, and the second equation gives the volume resulting from the mixture of two solutions when the strengths and densities of all the solutions and the volume of one of them are known, thus

$$\left.\begin{aligned} Q_1 &= rq \\ Q_2 &= Rq \end{aligned}\right\} \ldots\ldots\ldots\ldots (47)$$

21.—Similarity of Relations Involving Density, Strength, and Concentration.—In view of the definitions, there may be written, along with Equation (11),

$$\left.\begin{aligned} W_1\,p_1 + w\,p &= W_2\,p_2 \\ Q_1\,d_1 + q\,d &= Q_2\,d_2 \\ Q_1\,c_1 + q\,c &= Q_2\,c_2 \end{aligned}\right\} \ldots\ldots\ldots\ldots (48)$$

By dividing by w or q, there results

$$\left.\begin{aligned} f\,p_1 + p &= F\,p_2 \\ r\,d_1 + d &= R\,d_2 \\ r\,c_1 + c &= R\,c_2 \end{aligned}\right\} \ldots\ldots\ldots\ldots (49)$$

By a further division by p, d, or c, using the following notation

$$\left.\begin{aligned} P_x &= \frac{p_x}{p} \\ D_x &= \frac{d_x}{d} \\ C_x &= \frac{c_x}{c} \end{aligned}\right\} \ldots\ldots\ldots\ldots (50)$$

there results

$$\left.\begin{aligned} fP_1 + 1 &= FP_2 \\ rD_1 + 1 &= RD_2 \\ rC_1 + 1 &= RC_2 \end{aligned}\right\} \ldots\ldots\ldots\ldots (51)$$

Moreover, from Equation (6),

$$C = PD, \text{ etc.} \qquad (52)$$

To these should be added the analogous forms, Equations (40), and the other relations of Section 19. The similarity is apparent, and will be of frequent service in deriving other relations.

22.—Volumetric Equations Involving Densities and Concentrations. —The first two of these equations may be derived most simply by solving the last pair of Equations (51) for r and R, respectively.

Hence,

$$\left.\begin{aligned} r &= \frac{D_2 - C_2}{D_1\, C_2 - D_2\, C_1} \\ R &= \frac{D_1 - C_1}{D_1\, C_2 - D_2\, C_1} \end{aligned}\right\} \qquad (53)$$

Of course, to solve for quantities, we must use

$$\left.\begin{aligned} Q_1 &= qr \\ Q_2 &= qR \end{aligned}\right\} \qquad (54)$$

These equations are subject to the objection that the titration for c_1 will be more or less erroneous, as set forth fully in Sections 15 and 16. Hence C_1 and D_1, also, are likely to be in error by an amount larger than the permissible margin. This source of error may be removed by resort to special dilutions, also discussed in the two sections mentioned. An independent solution could be made by forming, from Equations (53), two equations corresponding to Equations (18), Section 13, after which the solution would proceed in a manner quite similar to that given in the remainder of Section 13; but the solution may be made more expeditiously by simply substituting in Equations (39), as follows:

$$\left.\begin{aligned} x &= \frac{p_2 - p'_2}{p - p_2} = \frac{\dfrac{c_2}{d_2} - \dfrac{c'_2}{d'_2}}{\dfrac{c}{d} - \dfrac{c_2}{d_2}} = \frac{C_2\, D'_2 - C'_2\, D_2}{D_2 - C_2}\,\frac{1}{D'_2} \\ y &= \frac{p_2 - p'_2}{p - p'_2} = \frac{\dfrac{c_2}{d_2} - \dfrac{c'_2}{d'_2}}{\dfrac{c}{d} - \dfrac{c'_2}{d'_2}} = \frac{C_2\, D'_2 - C'_2\, D_2}{D'_2 - C'_2}\,\frac{1}{D_2} \end{aligned}\right\} \qquad (55)$$

Thus, with the aid of the second of Equations (51), noting that $d_1 \div d = D_1$, etc.,

$$
\left.
\begin{aligned}
r &= \frac{r'}{1 + (r' D_1 + 1)\, x} \\
r &= \frac{R' \dfrac{D'_2}{D_1} - \dfrac{1}{D_1}}{1 + R' D'_2 x} \\
R &= \frac{r' D_1 + 1}{1 + r' D_1 y} \\
R &= \frac{R' \dfrac{D'_2}{D_2}}{1 + (R' D'_2 - 1)\, y}
\end{aligned}
\quad
\left\{
\begin{array}{l}
\text{By the method of special} \\
\text{dilutions the equations} \\
\text{involving densities and} \\
\text{concentrations. See} \\
\text{Equations (55).}
\end{array}
\right\}
\right\} \ldots (56)
$$

the last equation being the only one of the set free of D_1 as well as C_1.

The values for x and y are here supposed to be the last of those given in Equations (55), thus Equations (56) will be expressed entirely in terms of densities and concentrations. Of course, the reader will understand that the densities should be known, in fact, rather than be computed from the relations given in Table 1, Plates XLVIII, XLIX, or L, or Equations (4), which are supposed to apply only to solutions of pure sodium chloride in distilled water rather than to solutions of commercial salt in river water. However, in many cases, results will be sufficiently exact under the latter circumstances, where there is not much foreign matter carried in the water.

In case it is desired to check the salt content of the normal head-water from day to day, or test to test, as in Equations (28), we may, from the special dilutions, determine the value of C_1 in Equations (53) and then determine r or R by the same equations, but making use of the titrations of the tail-water instead of those of the special dilutions, just as in Equations (28); thus, from Equations (53),

$$\frac{D_2 - C_2}{r} = \frac{D_1 - C_1}{R}$$

or

$$C_1 = \frac{r D_1 - R D_2 + R C_2}{r}$$

$$= \frac{R C_2 - 1}{r}.$$

As $rD_1 - RD_2 = -1$, by Equations (51), hence, with special dilutions,

$$\left.\begin{aligned} C_1 &= \frac{R' C'_2 - 1}{r'} \\ r' &= \frac{D'_2 - C'_2}{D_1 C'_2 - D'_2 C_1} \quad \text{(Equations involving densities and concentrations. Values} \\ r &= \frac{D_2 - C_2}{D_1 C_2 - D_2 C_1} \quad \text{of } C_1 \text{ to check constancy of salt content.)} \\ \text{or} \quad R &= \frac{D_1 - C_1}{D_1 C_2 - D_2 C_1} \end{aligned}\right\} \ldots (57)$$

These equations may be obtained directly from Equations (28) by making the proper substitutions.

23.—Volumetric Shrinkage Coefficient.—It was explained in Section 5 that a shrinkage of volume takes place when two salt solutions of different densities are mixed. In many cases this shrinkage is relatively small, but there are cases where it would have to be taken into account when errors of less than 1% are of consequence, as, for example, where equal volumes of strong salt solution and distilled water are mixed.

It will appear presently that the total shrinkage of volume is only a relatively small fraction of the volume of the stronger of the two solutions which are mixed. Hence it will always be convenient to consider that the resultant volume is equal to the volume of the weaker solution plus a certain fraction of the original volume of the stronger solution.

Thus, if q is the volume of the stronger solution and Q_1 that of the weaker, the resultant volume may conveniently be written

$$Q_2 = Q_1 + kq \ldots\ldots\ldots\ldots\ldots\ldots (58)$$

where k is a positive fraction which will be found to approximate unity more or less closely in any case. It will be shown that the minimum value which k can have is approximately 0.96.

If Equation (58) is divided by q the result will be

$$R = r + k \ldots\ldots\ldots\ldots\ldots\ldots (59)$$

which is an equation of fundamental importance.

Now, from Equation (48),

$$Q_2 = Q_1 \frac{d_1}{d_2} + q \frac{d}{d_2}$$

$$= Q_1 + \left(Q_1 \frac{d_1}{d_2} - Q_1\right) + q \frac{d}{d_2}.$$

Therefore

$$\left.\begin{aligned} Q_1 + k\,q &= Q_1 + \left[\left(r\,\frac{d_1}{d_2} - r\right) + \frac{d}{d_2}\right] q \\ \text{or} \qquad k &= r\left(\frac{d_1}{d_2} - 1\right) + \frac{d}{d_2} \end{aligned}\right\} \text{.......}(60)$$

which gives the value of k in terms of the densities of the two solutions and their resultant mixture.

By analogy

$$k = r\left(\frac{c_1}{c_2} - 1\right) + \frac{c}{c_2} \text{..................}(61)$$

By taking the last two of Equations (49) with Equation (59), we have

$$\left.\begin{aligned} R &= r + k \\ Rd_2 &= rd_1 + d \\ Rc_2 &= rc_1 + c \\ &\text{or} \\ R &= r + k \\ RD_2 &= rD_1 + 1 \\ RC_2 &= rC_1 + 1 \end{aligned}\right\} \text{....................}(62)$$

from which, by eliminating R and r, we may derive a value for k in terms of densities and concentrations, thus

$$k = \frac{C_2 - C_1 - (D_2 - D_1)}{C_2\,D_1 + C_1\,D_2} \text{.................}(63)$$

which may also be easily obtained by noting from above that $k = R - r$, and then taking the difference between the two Equations (53).

Therefore k may be computed by any one of Equations (60), (61), or (63). In making the calculations, it is necessary, of course, to make use of the physical properties of the salt solution, as given in Table 1 or Plates XLVIII, XLIX, and L. It must be remembered that Equations (1), (2), (3), and (4) are limited to very dilute solutions.

By applying Equation (60) for various ratios of mixture, r, at different temperatures, Table 3 has been computed for two strengths of salt solution and for ratios of mixture from 1 to infinity. The correctness of the figures depends on the accuracy of Landolt and Börnstein's tables. If the table is in error, or if the salt solution contains impurities, the figures will include a corresponding error,

though this error, in many cases, will not be large. It may clarify matters to reiterate that the table has been computed for the water of the St. Lawrence River as it reached the power house during the turbine tests considered herein. The tabular values of k, therefore, will not exactly check the values obtained in Section 5.

TABLE 3.—VOLUMETRIC SHRINKAGE COEFFICIENT FOR THE WATERS OF THE ST. LAWRENCE RIVER DURING THE TURBINE TESTS CONSIDERED IN THIS PAPER.

(A) { Concentration of salt solution = 300 grammes per liter.
Concentration of canal water = 0.014 grammes per liter.

Temperature, in degrees, centigrade.	Values of r.							
	1	5	10	50	100	1 000	10 000	Infinity.
0°	0.9877	0.9748	0.9715	0.9672	0.9667	0.9663	0.9662	0.9662
10°	0.9898	0.9796	0.9771	0.9730	0.9725	0.9721	0.9720	0.9720
15°	0.9913	0.9818	0.9790	0.9745	0.9740	0.9736	0.9736	0.9735
20°	0.9916	0.9834	0.9821	0.9760	0.9755	0.9751	0.9751	0.9751
40°	0.9934	0.9862	0.9834	0.9804	0.9800	0.9796	0.9796	0.9796

(B) { Concentration of salt solution = 275 grammes per liter.
Concentration of canal water = 0.014 grammes per liter.

0°	0.9891	0.9777	0.9753	0.9715	0.9710	0.9706	0.9706	0.9706
10°	0.9913	0.9820	0.9798	0.9764	0.9760	0.9757	0.9756	0.9756
15°	0.9924	0.9842	0.9817	0.9777	0.9773	0.9770	0.9769	0.9769
20°	0.9927	0.9858	0.9836	0.9790	0.9786	0.9782	0.9782	0.9782
40°	0.9941	0.9887	0.9862	0.9829	0.9826	0.9822	0.9822	0.9821

NOTE.—The figure in the fourth decimal place is likely to be in error.

The relative shrinkage of volume on mixing the two salt solutions, as computed in Section 5, is given by the equation:

$$\frac{\Delta Q}{Q} = \frac{1 - k}{r + 1},$$

where ΔQ is the shrinkage of the mixture from the sum, Q, of the volumes of the two components before being mixed.

In computing the values of p_2, it will be of advantage to notice that

$$p_2 = \frac{r\, c_1 + c}{r\, d_1 + d} \qquad \text{(64)}$$

which may be derived from Equations (8), Section 9, by putting $w = q\, d$, $p = c \div d$, etc.

To facilitate interpolations for values of k, Fig. 1 has been prepared; its use will be understood without further explanation.

COEFFICIENT OF VOLUMETRIC SHRINKAGE
FOR.
MIXTURES OF SALT SOLUTION AND CANAL WATER.

The coefficient is the fraction by which the volume of the stronger solution must be multiplied to obtain the volume which when added to the volume of the weaker gives the volume of the subsequent mixture.

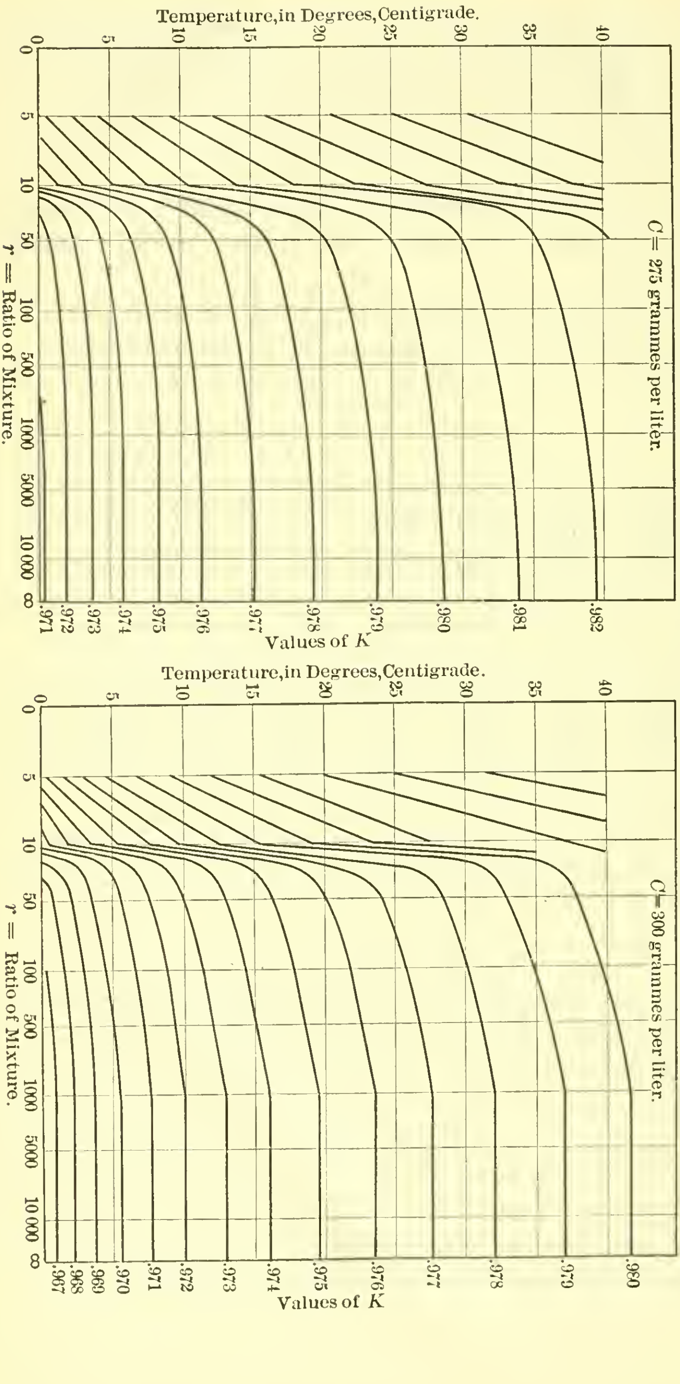


FIG. 1.

24.—Minimum Coefficient of Shrinkage.—It was stated in Section 23 that the least value of k is approximately 0.96. It will be seen from an inspection of Table 3 that the coefficient decreases when the density of the strong salt solution increases, when r increases, and when the temperature falls, the lowest tabular value being about 0.966. If we limit the temperature to zero, and take the strongest salt solution at that temperature, we may find, by Equations (60), (61), or (63), the value of k for any ratio of mixture. The least value will be for an infinite ratio, thus the problem is to find the limit of k when r is made to approach infinity as a limit.

Take, for example, Equation (63), noting that

$$C_1 = P_1 D_1, \text{ etc. (see Equations (6) and (50))} \quad \ldots\ldots (65)$$

$$D_2 = D_0 + n \frac{c}{d^2} P_1, \text{ etc. (see Equations (1) and (50))} \quad \ldots\ldots (66)$$

$$\begin{aligned} C_2 - C_1 &= P_2 D_2 - P_1 D_1 \text{ (from Equation 66)} \quad \ldots\ldots (67) \\ &= (P_2 - P_1) \left[D_0 + n \frac{c}{d^2} (P_2 + P_1) \right] \\ &= (P_2 - P_1)(D_2 + D_1 - D_0) \end{aligned}$$

$$D_2 - D_1 = n \frac{c}{d^2} (P_2 - P_1) \text{ (Equation (66))} \quad \ldots\ldots (68)$$

$$C_2 D_1 - C_1 D_2 = (P_2 - P_1) D_2 D_1 \text{ (Equation (65))} \quad \ldots\ldots (69)$$

Hence, substituting these values of $(C_2 - C_1)$, $(D_2 - D_1)$, and $(C_2 D_1 - C_1 D_2)$, there may be written, cancelling the common factor $(P_2 - P_1)$,

$$\left. \begin{aligned} k &= \frac{D_1 + D_2 - D_0 - n \frac{c}{d^2}}{D_2 D_1} \quad \text{(value of } n \text{ in Equations (1) and (2)).} \\ \text{or } k &= \frac{(d_1 + d_2 - d_0) d - n c}{d_1 d_2} \end{aligned} \right\} . (70)$$

where n depends on the temperature, as in Equations (1) and (2).

If, now, the ratios of mixture and dilution are made to approach infinity as a limit, the value of d_2 will change, approaching d_1 as a limit, while c and n are supposed to remain constant. Therefore, the limit of k is

$$\text{Lt. } k\Big]_{r = \infty} = \frac{(2 d_1 - d_0) d - n c}{d_1^2} \left\{ \begin{array}{l} \text{(salt solution, canal} \\ \text{water, zero degrees).} \end{array} \right\} \ldots (71)$$

It will be observed, also, that k will decrease when there is a decrease in the salt content of the water with which the salt solution is

mixed. Hence, to get the least value of the limit of k under the conditions supposed, let the mixture be of the strongest salt solution at 0° cent., with distilled water. That is, $d_1 = d_0$.
Therefore

$$\text{Limit } k\Big]_{r=\infty} = \frac{d_0\, d - n\, c}{d_0^{\,2}} \left\{ \begin{array}{l} \text{(salt solution, distilled} \\ \text{water, zero degrees.)} \end{array} \right\} \quad \ldots (72)$$

for the case supposed.

On reference to Tables 1 and 2 for numerical values, there may be found for a saturated solution of salt, of 26.4%, at 0° cent.,

$$\left. \begin{array}{l} n \;= 0.772 \\ c \;= 319.15 \\ d_0 = 0.99987 \\ d \;= 1.2089 \end{array} \right\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (73)$$

from which the least value of k is found to be

$$\left. \begin{array}{l} \text{Least value of } k \text{ for a mixture of} \\ \quad \text{saturated salt solution at } 0° \text{ cent.,} \\ \quad \text{with distilled water at the same} \\ \quad \text{temperature, } r = \infty\,. \end{array} \right\} = 0.9626 \ldots\ldots\ldots\ldots\ldots (74)$$

From the fifth column of Table 1 it will be found that the volume of distilled water required to make a liter of saturated salt solution at 0° cent. is 889.87. Hence, if Equations (2), Section 3, as derived from Landolt and Börnstein's tables are correct to the limit, it must follow that a molecule of salt always displaces a portion of the water in which it is dissolved, however dilute the solution may be. If the curves of Plate XLVIII approach a slope of 45° or more, when the concentration approaches zero, which does not appear to be the case, this statement would not be true.

Otherwise the limit of k, in Equation (74), would necessarily be 0.88987, the volume of distilled water in a cubic centimeter of saturated salt solution at 0° cent.

Of course, the same limit could be calculated from either Equations (60) or (61).

An actual observation of the value of k was made during the calibration of the solution tank by the chemical method on November 23d, 1914. (See Tables 34 and 35.) The tank was first filled with canal water to the point marked A on the calibration scale of the gauge-board, shown at A on Plate LII. A calibrator full of strong

salt solution containing about 265 grammes of salt per liter was then discharged into the tank and a thorough mixture was secured by running the circulating pump. The level of the mixed solution in the tank was then found to be at the point, *B,* on the gauge-board. At the left of Plate LII there is a full-sized sketch of the calibration scale.

Now, each division on the scale corresponds to the volumetric delivery of the calibrator. Thus, the space of 1.51 in. from Graduation 12 to Graduation 13 represents the volume delivered by the calibrator at this part of the scale. The rise of level due to the addition of a calibrator of salt solution, observed after a thorough mixture with the canal water originally in the tank, was 1.47 in., as shown on Plate LII. Therefore, the observed value of k in this experiment is $147 \div 151 = 0.973$.

By referring to Table 3, Section 23, and Fig. 1, the value of k, for a ratio of 65 and temperature of 5° cent., which are approximate data from the test, is seen to compare with the observed value above, as follows:

Observed value of k, November 23d, 1914........0.973

Interpolated tabular value for conditions of test...0.976+

which is within the limit of error probable in reading the elevation of the surface of the solution with the calibration scale. It is not safe, however, to conclude from this one observation that a molecule of salt always displaces a portion of the water in which it is dissolved. The conditions of the test, too, are far removed from the limiting conditions supposed to exist when deriving the limit in Equation (74).

25.—Successive Dilutions.—It has been seen that it is very serviceable, and conducive to accuracy, to dilute the salt solution so as to lead to titrations of samples of the same concentration. It is sometimes convenient to dilute the tail-water to a certain degree of concentration for the same purpose. Hence, it is important to have the relations of the quantities under such circumstances expressed in algebraic form.

From Equations (62), Section 23, we have, for a first dilution, symbolized by R', r', c'_2, etc.,

$$c' = \frac{r'\, c_1 + c}{R'} \quad \text{.........................(75)}$$

and for a second dilution of this tail-water with a new portion of normal head-water, symbolized by R'', r'', etc.

$$c'' = \frac{r'' \, c_1 + c'}{R''} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (76)$$

Substituting the value of c' from Equation (75), we derive

$$c'' = \frac{r'' \, c_1 + \dfrac{r' \, c_1 + c}{R'}}{R''} = \frac{R' \, r'' \, c_1 + r' \, c_1 + c}{R' \, R''}$$

Therefore,

$$c'' = \frac{(R' \, r'' + r') \, c_1 + c}{R' \, R''} \quad \ldots\ldots\ldots\ldots (77)$$

Thus, the whole operation results in a solution which is exactly the same as though the ratio of mixture of normal head-water and salt solution had been $R' \, r'' + r'$ and the ratio of dilution $R' \, R''$. Denoting the ratio of mixture and ratio of dilution of the resulting solution by r and R, respectively, the statement may be expressed as follows:

$$\left.\begin{aligned} r &= R' \, r'' + r' \\ \text{and} \qquad R &= R' \, R'' \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (78)$$

We may, of course, also write

$$r = (r' + k') \, r'' + r' = r'r'' + k'r'' + r' \quad \ldots\ldots\ldots\ldots (79)$$

and

$$\begin{aligned} r + k &= (r' + k') \, (r'' + k'') \\ &= r'r'' + k'r'' + r'k'' + k'k'' \quad \ldots\ldots\ldots\ldots (80) \end{aligned}$$

We may also deduce relations between k, k', and k''. Thus

$$\left.\begin{aligned} k &= R - r \\ &= R'R'' - R'r'' - r' \\ &= R'k'' - r' \\ &= (r' + k') \, k'' - r' \\ &= k'k'' + r'k'' - r' \\ &= k'k'' + r'(k'' - 1) \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots (81)$$

26.—It will aid the comprehension considerably to deduce the relations of Equations (78) by direct reasoning rather than by formulas. Take, for example, a volume, x, of salt solution and dilute with a volume r' times larger, that is with a volume, $r'x$. Then, by our theory and definitions, the resulting mixture will occupy a volume $k'x +$ $r'x = (r' + k')x$.

Hence 1 liter of the mixture contains $x \div (r' + k')x = 1 \div$ $(r' + k')$ of original salt solutions, and $r'x \div (r' + k')x = r' \div$

$(r' + k')$ of normal head-water, the resulting volume of this liter, of course, being

$$\frac{k'}{r' + k'} + \frac{r'}{r + k'} = 1 \text{ liter.}$$

If, now, a second dilution is made by taking a volume, y, of the first mixture and diluting with a volume r'' times larger, that is, with a volume, $r''y$, we shall have taken the equivalents of

$$\frac{y}{r' + k'} \text{ of original salt solution}$$

and

$$\frac{y\,r'}{r' + k'} \text{ of normal head-water}$$

to mix with $r''y$ of normal head-water.

The resulting mixture, therefore, is the same as though

$$\frac{y}{r' + k'} \text{ of original salt solution and } \frac{y\,r'}{r' + k'} + r''\,y$$

of normal head-water had been mixed in one operation.

Therefore,

$$r = \frac{\left(\frac{r'}{r' + k'} + r''\right) y}{\frac{y}{r' + k'}} = (r' + k')\,r'' + r'$$

or

$$r = R'\,r'' + r'$$

the same as in Equations (78), which was proved by algebraic substitutions.

27.—Successive Dilutions in Laboratory.—Suppose now, that, for convenience in laboratory work, the tail-water is diluted in such a manner that r'' liters of normal head-water are mixed with 1 liter of tail-water before taking samples for evaporation. That is, the ratio of mixture is r'' and the ratio of dilution is $r'' + k'' = R''$. Then, if the diluted tail-water is evaporated in the usual manner and the test is reduced as though this had been the actual tail-water, the ratios of mixture and dilution for the actual tail-water can be easily computed from the results, thus,

$$\left.\begin{aligned} r' &= r - R'\;\,r'' \\ r' &= r - \frac{R}{R''}\,r'' \end{aligned}\right\} \text{(See Equations (78))} \ldots\ldots\ldots (82)$$

which latter is in terms of the laboratory dilution and final ratios of mixture and dilution computed from the modified laboratory procedure indicated above.

In a similar manner, we may have, directly from Equations (79),

$$\left.\begin{aligned} r' &= \frac{r - k' r''}{1 + r''} \\ R' &= r' + k' = \frac{r - k' r''}{1 + r''} + k' \\ &= \frac{r}{1 + r''} + \frac{k'}{1 + r''} \end{aligned}\right\} \qquad (83)$$

but such forms are not entirely independent of the first dilution, as they involve k', or other functions of r' or k'.

Again, from Equations (78),

$$R' = \frac{R}{R''} \qquad (84)$$

which may be used to compute R' when R has been computed from the results of titrations of the samples of diluted tail-water and R'' has been observed. In terms of these same quantities, r' may be expressed as follows, from Equations (82),

$$\left.\begin{aligned} r' &= r - \frac{R}{R''}(R'' - k'') \\ &= r - R + \frac{R}{R''} k'' \\ &= \frac{R}{R''} k'' - k \\ &= \frac{R\,k'' - k\,R''}{R''} \end{aligned}\right\} \qquad (85)$$

28.—Inverse Dilutions.—Considerable space has been devoted to the theory of mixtures of salt solutions. The study has been necessary to educate the writer sufficiently to enable him to offer very simple proofs of the fundamental volumetric equations depending on ratios of mixture and dilution only. Before proceeding with the discussion it will be well, however, to recall the relations existing between the tail-water and special dilution samples which have been prepared in the laboratory.

The tail-water and special dilution are both mixtures of salt solution with normal head-water, the respective ratios of mixture being r and r' and the respective ratios of dilution R and R'.

As it is the object of the special dilution to reproduce a solution as nearly as possible like the tail-water, we shall always have, more or less closely,

$$R \text{ approximates } R'$$
$$\text{and}$$
$$r \text{ approximates } r'$$

If it happens that the special dilution is stronger in salt than the tail-water, then $R > R'$ and $r > r'$; but, if the contrary is true, $R' > R$ and $r' > r$. In the former case salt solution may be added to the tail-water until the mixture is just like the special dilution, and in the latter case salt solution may be added to the special dilution until it is just like the tail-water. In each case the quantity of salt solution necessary will depend entirely on the values of R and R', or of r and r', and the quantity of the tail-water, or special dilution, with which it is mixed. In either case the necessary ratio of dilution of salt solution will be indicated by $\boldsymbol{R}''$.

Such a dilution of salt solution by another solution would convert the latter into a stronger one, whereas, a dilution of the solution with normal head-water would convert it into a weaker one. In this sense the dilution of salt solution with tail-water, or special dilution, will be called an inverse dilution.

29.—Volumetric Equations Involving Ratios Only.—Suppose, first, that the tail-water is weaker in salt than the special dilution, or $R > R'$ and $r > r'$. Let $\boldsymbol{R}''$ be the ratio of dilution necessary to convert tail-water into special dilution by adding salt solution. Take sufficient tail-water to make the resulting mixture have a volume, R', and therefore contain r' of normal head-water. This must be $\frac{r'}{r} R$ of tail-water, which necessarily contains $\frac{r'}{r}$ of salt solution, as R of tail-water contains r of normal head-water and the resultant mixture must contain only r'. It is also apparent from the premises that the requisite quantity of salt solution to mix with the tail-water is $R' \div \boldsymbol{R}''$ and that the total quantity of salt solution in the resulting mixture must be unity. Therefore,

$$\frac{R'}{\boldsymbol{R}'} + \frac{r'}{r} = 1$$

from which

$$r = \frac{r'}{1 - \frac{R'}{R''}} \dots\dots\dots\dots\dots\dots\dots\dots (86)$$

30.—The tail-water still being weaker than the special dilution, let r'' be the ratio of mixture necessary to convert the former into the latter by adding salt solution. Take a volume, R, of tail-water, which contains r of normal head-water and 1 of salt solution. It will require $R \div r''$ of salt solution to reduce the mixture to the conditions of special dilution, as tail-water is being mixed with the salt solution and r'' is the ratio of mixture. Thus, the resulting mixture contains $1 + \frac{R}{r''}$ of salt solution and r of normal head-water. The resulting mixture, however, is exactly the same as the special dilution. Therefore,

$$\frac{r}{1 + \frac{R}{r''}} = r' \dots\dots\dots\dots\dots\dots\dots\dots (87)$$

31.—From Equations (86) and (87) it is clear that

$$\left(1 + \frac{R}{r''}\right)\left(1 - \frac{R'}{R''}\right) = 1 \dots\dots\dots\dots\dots\dots (88)$$

and that the quantity of salt solution required to reduce a solution the ratio of mixture of which with the same salt solution is r to a stronger one having a ratio of r' is such that

$$\left.\begin{aligned} R'' &= \frac{R'}{1 - \frac{r'}{r}} \\ \text{and} \qquad r'' &= \frac{R}{\frac{r}{r'} - 1} \end{aligned}\right\} (r > r') \dots\dots\dots\dots (89)$$

32.—*Reciprocal Dilutions.*—It will be seen by reference to Section 25 that the second dilution with normal head-water there discussed converts special dilution into tail-water; and the dilution with salt solution examined in Sections 29 and 30 returns the mixture to its original condition as regards its relative composition. A pair of dilutions thus related may be called a pair of reciprocal dilutions.

It is clear from Section 25, Equations (78), that $R'r''$ is the quantity of normal head-water necessary to convert R' of special dilution to the condition of tail-water, and, from Section 30, that $R \div r''$ of salt solution will return the mixture to its original condition of special dilution in which the ratio of mixture is r'. Therefore the quantity, $R'r''$, of normal head-water and the quantity, $R \div r''$, of salt solution must be mixed in the same ratio as the special dilution, or

$$\left.\begin{aligned} \frac{R'\,r''}{\dfrac{R}{r''}} &= r' \\ \text{and} \qquad r''\,r'' &= R''\,r' \end{aligned}\right\} \ldots\ldots\ldots\ldots\ldots\ldots (90)$$

From Section 29, if $\frac{r'}{r}R$ of tail-water is mixed with $\frac{R'}{R''}$ of salt solution, the mixture will be of volume, R', will contain a unit volume of original salt solution, and r' of normal head-water. Consequently, the $\frac{r'}{r}R$ of tail-water must have contained r' of normal head-water and $1 - \frac{R'}{R''}$ of salt solution. Therefore, using Equations (78),

$$\frac{r - R'\,r''}{1 - \dfrac{R'}{R''}} = r$$

from which

$$R''\,r'' = r \qquad (r > r') \ldots\ldots\ldots\ldots\ldots (91)$$

Expanding the product in Equation (88), there results

$$\frac{R}{r''} - \frac{R'}{R''} - \frac{R'\,R}{R''\,r''} = 0 \ldots\ldots\ldots\ldots\ldots\ldots (92)$$

Hence,

$$\left.\begin{aligned} \frac{R}{r''} &= \frac{\dfrac{R'}{R''}}{1 - \dfrac{R'}{R'}} \\ & \qquad\qquad (r > r') \\ \frac{R'}{R''} &= \frac{\dfrac{R}{r''}}{1 + \dfrac{R}{r''}} \end{aligned}\right\} \ldots\ldots\ldots\ldots\ldots (93)$$

which may also be written

$$\left.\begin{aligned} \frac{R}{\boldsymbol{r}''} &= \frac{1}{\dfrac{\boldsymbol{R}''}{R'} - 1} \\ \frac{R'}{\boldsymbol{R}''} &= \frac{1}{\dfrac{\boldsymbol{r}''}{R} + 1} \end{aligned} \quad (r > r') \right\} \dots\dots\dots\dots (94)$$

From Equations (93) we have, as $\boldsymbol{R}'' - \boldsymbol{r}'' = \boldsymbol{k}''$,

$$\boldsymbol{k}'' = \frac{R' r - R r'}{r - r'} \dots\dots\dots\dots (95)$$

which reduces easily to

$$\boldsymbol{k}'' = \frac{k' r - k r'}{r - r'} \dots\dots\dots\dots (96)$$

As the greater r must be associated with the smaller k (see Table 3 or Fig. 1), it must follow that

$$\boldsymbol{k}'' > k \text{ and } \boldsymbol{k}'' > k'.$$

When $\boldsymbol{r}$ and r' are nearly equal, and relatively large, it is clear that $\boldsymbol{k}'' = k = k'$, practically. Indeed, the limit of $\boldsymbol{k}''$ for fixed temperatures is $k' - r' \dfrac{d\,k'}{d\,r'}$, when r approaches r' as a limit. Fig. 1 shows that $r \dfrac{d\,k}{d\,r}$ is small, relatively to k, for all likely values of r.

33.—Since the discussion of Section 25, it has been assumed generally that the tail-water was a weaker solution than the special dilution, that is, that $r > r'$. If the contrary is true, and $r' > r$, then it would be necessary to dilute the tail-water with normal head-water to convert it to a solution like the special dilution, and to introduce salt solution into the special dilution to reproduce the conditions of tail-water. The foregoing equations for the successive and inverse dilutions would still hold, all the reasoning applying exactly as before, provided that r' and r, and R' and R are mutually interchanged in the equations, while the definitions of $\boldsymbol{R}''$, $\boldsymbol{r}''$, R'' and r'' would require a corresponding modification agreeable to the changed sense of the operations.

34.—Volumetric Equations Involving Concentrations Only.—From Equations (62),

$$\left.\begin{aligned} R &= r + k \\ R\,C_2 &= r\,C_1 + 1 \end{aligned}\right\} \dots\dots\dots\dots (97)$$

from which,

$$\left.\begin{aligned} r &= \frac{1 - k\,C_2}{C_2 - C_1} \\ R &= \frac{1 - k\,C_1}{C_2 - C_1} \end{aligned}\right\} \quad \text{(98)}$$

in terms of concentrations which are relative with respect to the concentration of the strong salt solution.

In terms of the actual concentrations,

$$\left.\begin{aligned} r &= \frac{c - k\,c_2}{c_2 - c_1} \\ R &= \frac{c - k\,c_1}{c_2 - c_1} \end{aligned}\right\} \quad \text{(99)}$$

which are derived from Equations (98) by multiplication by c. Thus

$$c_1 = c\,C_1.$$

These forms, however, are subject to the errors of titration which affect c_1 relatively to c and c_2, as has been explained in Sections 15 and 16. In order to overcome this objection, the method of special dilutions is available, as described in the sections referred to and elsewhere in this paper.

35.—The preceding discussions enable us to write out at once the equations pertinent to the method of special dilutions and involving only the concentrations and one coefficient of shrinkage. These equations are the most convenient where the samples are measured volumetrically, but, as has been explained in Section 10, the equations of Section 14 are much better adapted to the requirements of turbine testing. Resuming the assumption that $r > r'$, the definitions of Sections 29 and 30 require that

$$\left.\begin{aligned} r'' &= \frac{c - k''\,c_2}{c'_2 - c_2} \\ & \qquad\qquad (r > r') \\ R'' &= \frac{c - k''\,c'_2}{c'_2 - c_2} \end{aligned}\right\} \quad \text{(100)}$$

$$0.96 < k'' < 1$$

which are the same in form as Equations (99).

But, if the assumption is that $r' > r$, we must have

$$\left.\begin{aligned} r'' &= \frac{c - k''\,c_2}{c_2 - c'_2} \\ & \qquad\qquad (r' > r) \\ R' &= \frac{c - k''\,c'_2}{c_2 - c'_2} \end{aligned}\right\} \quad \text{(101)}$$

Hence, changing the assumption is equivalent to substituting $-\boldsymbol{r}''$ for $\boldsymbol{R}''$ and $-\boldsymbol{R}''$ for $\boldsymbol{r}''$ in Equations (100) or (101).

Now, under the assumption that $r > r'$, and consequently that $c'_2 > c_2$ we have, by Equations (86) and (100),

$$\left.\begin{aligned} r &= \frac{r'}{1 - \dfrac{R'}{\boldsymbol{R}''}} \\ \text{or} \qquad & \qquad\qquad\qquad (c'_2 > c_2) \\ r &= \frac{r'}{1 + R' \dfrac{c_2 - c'_2}{c - \boldsymbol{k}'' c_2}} \end{aligned}\right\} \ldots\ldots\ldots\ldots(102)$$

But, under the assumption that $r' > r$, and consequently that $c_2 > c'_2$, we have, interchanging r and r', and putting R' for R, by Equations (87) and (101),

$$\left.\begin{aligned} r &= \frac{r'}{1 + \dfrac{R'}{\boldsymbol{r}''}} \\ \text{or} \qquad & \qquad\qquad\qquad (c_2 > c'_2) \\ r &= \frac{r'}{1 + R' \dfrac{c_2 - c'_2}{c - \boldsymbol{k}'' c_2}} \end{aligned}\right\} \ldots\ldots\ldots\ldots(103)$$

We have, by Equations (93),

$$R = \frac{\dfrac{R'}{1 + \dfrac{\boldsymbol{k}''}{\boldsymbol{r}''}}}{1 - \dfrac{R'}{\boldsymbol{R}''}}$$

which reduces easily to

$$\left.\begin{aligned} R &= \frac{R'}{1 + \dfrac{\boldsymbol{k}'' - R'}{\boldsymbol{r}''}} \\ \text{or} \qquad & \qquad\qquad\qquad (c'_2 > c_2) \\ R &= \frac{R'}{1 - (\boldsymbol{k}'' - R') \dfrac{c_2 - c'_2}{c - \boldsymbol{k}'' c'_2}} \quad \text{(see Equations (100))} \end{aligned}\right\} \ldots (104)$$

But when $r' > r$, we have, by substituting r for r', R for R', and *vice versa*, by Equations (93) and 101),

$$\left.\begin{aligned} R &= \frac{\dfrac{\mathbf{R''}}{\mathbf{r''}} R'}{1 + \dfrac{R'}{\mathbf{r''}}} \\ \text{or} \qquad & \qquad\qquad (c_2 > c'_2) \\ R &= \frac{R'}{1 - (\mathbf{k''} - R') \dfrac{c_2 - c'_2}{c - \mathbf{k''}\, c'_2}} \end{aligned}\right\} \ldots\ldots (105)$$

Therefore, it is proved generally, whether $c_2 > c'_2$, or the contrary, that the general equations for r and R by the method of special dilutions are

$$\left.\begin{aligned} r &= \frac{r'}{1 + (r' + k') \dfrac{c_2 - c'_2}{c - \mathbf{k''}\, c_2}} \\ r &= \frac{R' - k'}{1 + R' \dfrac{c_2 - c'_2}{c - \mathbf{k''}\, c_2}} \qquad \text{(General formulas, method of special dilutions.)} \\ & \qquad\qquad (1 > \mathbf{k''} > 0.96) \\ r &= \frac{r'}{1 + R' \dfrac{c_2 - c'_2}{c - \mathbf{k''}\, c_2}} \\ R &= \frac{R'}{1 + (R' - \mathbf{k''}) \dfrac{c_2 - c'_2}{c - \mathbf{k''}\, c'_2}} \\ R &= \frac{r' + k'}{1 + (r' + k' - \mathbf{k''}) \dfrac{c_2 - c'_2}{c - \mathbf{k''}\, c'_2}} \end{aligned}\right\} \ldots (106)$$

36.—Equations (106) may, of course, be deduced in numerous other ways, for example, by making use of Equations (27) of Section 14, Equations (38) of Section 18, Equations (53) of Section 22, and the relations of Equations (100) and (101) of Section 35.

37.—Constancy of Salt Content in Head-Water.—Similar to Equations (28) of Section 14, and Equations (45) of Section 20, are the equations derived from either Equations (62) of Section 23, or Equations (98) and (99) of Section 34, when special dilutions are

made up in the laboratory, and the normal head-water titration is calculated instead of observed, thus,

$$\left.\begin{array}{ll} c_1 = \dfrac{R' c'_2 - c}{r'} & \text{(Value } c_1 \text{ to check constancy of head-water content of salt.)} \\[2ex] r = \dfrac{c - k\, c_2}{c_2 - c_1} & \text{(General formulas for method of special dilutions.)} \\[2ex] R = \dfrac{c - k\, c_1}{c_2 - c_1} & (1 > k > 0.96) \end{array}\right\} \cdots\cdots (107)$$

The special dilutions eliminate the unbalanced evaporations for c_1, as compared with the evaporations for c_2.

Of course, it has not been forgotten that we may substitute v for c in these equations, so as to introduce the titrations per liter of the various solutions, instead of the concentrations. Indeed, by slight modifications of these formulas, the actual titrations, t, t_2, etc., may be substituted, as was illustrated in Equations (33) of Section 16, and as will be shown in the discussion of the theory involved in Equations (27), (28), (39), (44), (45), (106), (107), (108), and others of similar nature. This discussion appears in Section 45.

38.—Volumetric Equations Involving Densities Only.—Owing to the relations discussed in Section 21, there may be written, by analogy,

$$\left.\begin{array}{ll} r = \dfrac{r'}{1 + R' \dfrac{d_2 - d'_2}{d - \mathbf{k}'' d_2}} & (1 > \mathbf{k}'' > 0.96) \\[3ex] R = \dfrac{R'}{1 + (R' - \mathbf{k}'') \dfrac{d_2 - d'_2}{d - \mathbf{k}'' d'_2}} & \end{array}\right\} \cdots (108)$$

which might be used for the purpose of determining volumes of discharge when facilities are at hand for determining densities with the requisite precision.

39.—Color.—Instead of d relating to the specific quantity of matter, or quantity of chemical, present, it may relate to the intensity of any other property, such, for example, as color, which can be diffused throughout the volume of the liquid, the term, d, being in that case a measure of the quantity of coloring matter necessary to give the liquid a measurable or comparable tint. In such case the tints due

to d_2 and d'_2 can be made to agree exactly, so that the second term of the denominator vanishes and we have simply

$$R = R' \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (109)$$

where R' is the only quantity necessary to measure, the colors of the tail-water and special dilutions being exactly matched. A colorimeter of special construction will evidently facilitate the determinations of R', and therefore of R.

40.—Electro-Metric Methods.—Time will not be taken up in entering into the great varieties of methods which may be used in gauging by "dosing" the feed-water of a turbine. Suffice it to state that electro-metric methods are also being studied at the present time in various parts of the world, notably by Dr. Mellet and P. Dutoit, in Switzerland. The writer also has had made, at his suggestion, some tests of this character which seem promising.

Dr. Mellet's pamphlet, "Les Jaugeages par Titration Physico-Chemique," an extract from *Bulletin Technique* de la Suisse Romande, February 10th, 1915, page 25, gives the results of several gaugings by this method, with an account of the nature of the processes.

The writer's method of applying the conductivity test is: First, determine the conductivity of the tail-water; secondly, prepare a special dilution of the same salt solution which was used in the test with normal head-water, approximating as nearly as possible the actual ratio during the test; thirdly, with the same electrodes and the same electrical equipment used with the tail-water, determine whether the conductivity of the special dilution is greater, or less, than that of the tail-water; fourthly, measure from a burette the quantity of salt solution, or normal head-water, as the case may be, necessary to equate exactly the conductivity of the special dilution to that of the tail-water, the burette discharging directly into the special dilution, which is constantly stirred, until the electrical indicator shows that the two conductivities are equal; fifthly, the known ratio of dilution of the special dilution, determined from the measured quantities of the components, is then the required ratio of dilution for the test; sixthly, this ratio multiplied by the rate of discharge of the salt solution during the test gives the discharge of the turbine. Of course, the method of weights as against the method of volumes has the same advantages when measurements are made electrically as when they are made chemically.

PART II.

ERRORS.

SUMMARY.

41.—Part II discusses at length the errors of the method and the procedure for their elimination. Corrections have been determined and tabulated for the method of unbalanced evaporations.

Incidentally, the method of balanced evaporations, or special dilutions, leads to an independent determination of the error produced by unbalanced evaporations, and proves that the writer's method of correcting the results by the latter method is correct.

A system of group titrations is devised, for the purpose of eliminating a large variety of errors.

The effect of non-uniformity of mixture of the chemical with the feed-water is shown to be less than many have heretofore supposed. This was shown in the tests by throwing the heavier concentration first to one side of the raceway and then to the other. The properly weighted concentrations give results which plot on a line, while the unweighted concentrations do not exhibit such discrepancies as would be a serious matter in tests of ordinary precision. In the best results, on Unit No. 13, the arithmetical mean concentrations would have served almost as well as the weighted mean.

The centrifugal pump is proved to be an excellent mixer of salt solution and canal water. Tests on such pumps can be executed with the utmost precision by the chemical method. Indeed, it is doubtful whether a better method can be found for the testing of pumps.

The weights for averaging the tail-water concentrations were determined by current meters in the tail-race. The still-water ratings, without correction, were used, but it is likely that some better method can be devised for determining weights.

The difficulties encountered in securing representative tail-water samples are related, with the means adopted for removing all uncertainty in this respect.

The method of designing and installing efficient sprinkler mixing systems is discussed, with the results of experiments bearing on sprinklers and perforations.

Pipette calibrations of an unusual character are given which show the combined and separate effects of temperature and concentration of

salt solution on the discharge of these instruments. It has been the practice, even by the United States Bureau of Standards, to calibrate a pipette at one temperature and then to assume that the changes in discharge due to changes in temperature are equal to the corresponding changes in volume of the pipette, thus disregarding the effects on the discharge of those properties of the contained liquids which change with the temperature.

It is shown herein that the change in discharge due to a change in temperature is entirely distinct from the change in volume of the pipette due to the same change in temperature and that the combined effects of viscosity, surface tension, cohesion, and capillary action in general affect the discharge to a very much greater extent. The total change in discharge is several times the change in volume of the pipette. Isograms giving the discharge of pipettes for salt solutions of all concentrations at various temperatures are given as a ready means for determining the discharge.

The calibrations of flasks are given at all working temperatures, and rules are stated for effecting a perfect mixture by shaking the flasks containing mixtures of salt solutions and water. It does not appear that it is generally known how many times an ordinary liter flask must be inverted and shaken to secure a uniform mixture. A failure of the chemists to secure accurate mixtures during the early tests necessitated a special investigation to determine this question.

It is proved that there is no systematic error of any consequence in the chemical procedure developed and used during the tests. This conclusion is proved by the close agreement between the chemical and gravimetric calibrations of the salt solution tanks. The tests are of greater value on account of the unfavorable conditions under which they were executed than they would have been under the more propitious conditions of an extremely painstaking laboratory process.

It is shown that the wooden salt solution tank used during the tests was remarkably constant in volume, probably not changing its capacity by one-tenth of 1% during the entire season.

A method of eliminating the error introduced into the computed efficiency of a turbine by reason of an error in the density of the feed-water is given, and also a method for obviating the necessity for determining the density of the feed-water. That is, the density of feed-water has been entirely eliminated from the calculation.

The weight of the salt injected into the feed-water may be neglected in most cases.

The question is raised as to the correct method of determining the head acting on a turbine. It is shown that the velocity-head should be taken into account, and simple methods for its determination are discussed.

The effect on the tail-race concentration, and therefore on turbine efficiency, of variations of discharge and power during a test have been determined, and are shown to be negligible where the variations are less than certain limiting variations.

The relation of volume to time averages is discussed, so that, with a time-average concentration, a volume average may be computed, and *vice versa.* Time averages are shown to be accurate enough for most purposes. A device is suggested for determining volume averages.

The sources of error in chemi-hydrometry may be classified as follows:

1.—*Initial Content of Chemical.*—There may be chemicals in the stream initially which react with the standard; or the dosing chemical may be affected by them.

2.—*Variability of Initial Content.*—The initial content of a chemical may be variable, and this variation may be appreciable from minute to minute.

3.—*Variability of Standard Chemical.*—The character of the standard, and consequently the equivalent weights of other chemicals, may change during a series of chemical analyses.

4.—*Indicator.*—The indicator used in volumetric analyses may act on the standard chemical. This is the case to an appreciable extent when chromate of potassium and silver nitrate are used.

5.—*Excess of Standard.*—An additional excess of standard chemical will usually be required to effect a definite determination of the end of the reaction.

6.—*Distribution of Chemical.*—The chemical is not likely to be distributed uniformly throughout the mass of the liquid to be measured, either as to time or place, except in the case of relatively small quantities of liquid.

7.—*Temperature Effects.*—The temperature during a test, or series of tests, and at the time of the ensuing treatment of samples in the laboratory, may be different at different times and places, thus introducing changes in the capacities of the measuring instruments, and corresponding changes in the volumes and properties of the various solutions used.

8.—*Changes in Calibrations.*—There may be other circumstances which cause changes in the calibrations of the various instruments, such for example as a change in the volume of the tank from which the dosing chemical is discharged and measured.

9.—*Time.*—Even if the watches have been carefully regulated and rated, there may be errors in observing, such, for example, as the personal error of an observer.

10.—*Density of Feed-Water.*—In turbine testing an error will be made if the density of the feed-water entering into the efficiency calculations is in error. This density may be determined or the error may be eliminated by the method of counter errors, explained in the sequel.

11.—*Weight of the Chemical.*—A similar error to that preceding will be made if the weight of the chemical injected into the head-water is not taken into account. However, in most cases this will be inappreciable.

12.—*Additions or Abstractions of Chemical.*—There may be additions or abstractions of chemical between the dosing and sampling stations, due either to natural causes, such as mineral springs, or to artificial causes, such as the operation of industries between the stations.

13.—*Additions or Abstractions of Water.*—There may be additions or abstraetions of water between the stations due to natural or artificial causes.

14.—*Error in Tables.*—There may be errors in the physical constants used in the computations. For example, there may be errors in Table 1.

15.—*Errors in Theory.*—The theory may not be complete, or it may be fundamentally wrong. For example, it is shown later that there is an error in using Equations (17) of Section 11, without correcting v_1 for the error of titration, whereas,

Equations (27) and (28), Sections 13 and 14, are correct in this respect when the data are correctly observed.

16.—*Air in Feed-Water.*—On referring to Section 73, it will be seen that the density of the feed-water may be eliminated. It is also clear that the chemical method of measuring discharge does not include the volume, or weight, of any air, which may be entrained in the water temporarily as it passes the turbine.

42.—Net Error Due to Imperfect Theory.—In discussing the foregoing errors, it will be of advantage first to take up the net error due to imperfect theory, after which a detailed examination of the separate errors will be made.

In order to examine the theory, let

c_s = concentration of the standard chemical;

v = volume of the standard required to titrate a unit volume of sample;

V = volume of sample titrated;

e = equivalent of the dosing chemical, salt for example, per gramme of the standard chemical. In theory this will be the ratio of molecular weights, but in practice it will vary from such a ratio, due to various sources of error;

t = actual titration of a sample which is not exactly one unit in volume, thus $v = t \div V$, $v_2 = t_2 \div V_2$.

Then, for example, theoretically, we must have such equalities as

$$\left.\begin{aligned} c_2 &= \frac{t_2\, c_s\, e}{V_2} \\ \text{or} \qquad Q &= q\,\frac{c - k\,\dfrac{t_2\, c_s\, e}{V_2}}{\dfrac{t_2\, c_s\, e}{V_2} - \dfrac{t_1\, c_s\, e}{V_1}} \end{aligned}\right\} \qquad (110)$$

where e is strictly the ratio of chemical equivalents; but, in actual cases, it will seldom be found that such expressions constitute actual equalities; one side or the other will be the greater. Such a failure of theory may result from one or more sources of error.

It must not be supposed, therefore, that the ratio of chemical equivalents, e, deduced from the atomic weights of the elements involved, can be used with any degree of reliability in practical cases. Even where the chemicals are pure, there will be required a certain

excess of standard chemical in order to act on the indicator, and this excess will vary with circumstances. This is equivalent to saying that the practical value of e is variable to a slight extent.

It is necessary, therefore, to determine by actual observation what the net effect of the errors is, in any particular case, so that the observed discharge can be corrected; or, measures must be adopted so that the errors will be eliminated or reduced to inappreciable magnitudes.

43.—Practical Determination of Net Error of Titration for Equations (14), (17), (46), (53), (98), and (99).—Take Equations (99), Section 34, for example. These are virtually the same as Equations (53), Section 22, Equations (46), Section 20, or Equations (14) and (17), Section 11. If we substitute $Q_1 \div q$ for r, and $(Q_1 + kq) \div q$ for R, we have

$$Q_1 c_1 + qc = (Q_1 + kq) c_2 \text{..............} (111)$$

In terms of silver nitrate, or other chemical, this may be written

$$Q_1 v_1 + qv = (Q_1 + kq) v_2 \text{..............} (112)$$

v, v_1, and v_2 being the quantities of silver nitrate required per liter of the samples as used in the tests. Hence, if t, t_1, and t_2 are the actual quantities of silver nitrate required for the corresponding samples, we shall have

$$v = \frac{t}{q}, \quad v_1 = \frac{t_1}{Q_1}, \text{ and } v_2 = \frac{t_2}{Q + k\,q} \text{..........} (113)$$

Substituting these values in Equation (112), we derive

$$t_1 + t = t_2 \text{.....................} (114)$$

as might naturally be inferred without reference to Equation (112). That is to say, if three samples are prepared, consisting, respectively, of Q_1 of normal head-water, q of salt solution, and a mixture of Q_1 of normal head-water and q of salt solution, we should have the relation shown by Equation (114) between the titrations, because the last of the three samples is simply a mixture of samples exactly like the first two.

But, if the relation of Equation (114) fails to hold in actual trial, the fact will prove that the theory is incomplete, involving a systematic error, or that the methods do not eliminate the errors of titration. Moreover, if the samples, Q_1 and q, are of the same size as those

corresponding in the actual test, the difference between $(t_1 + t)$ and t_2 will determine the error due to the titration of samples of these sizes and concentrations.

A number of tests were made in this manner to determine whether the theory involved in the foregoing equations was exact or only approximate, with the result that the latter was shown to be the case. Table 4 exhibits the results which have been collected for this purpose. It must be impressed on the mind that these data are not derived from refined laboratory experiments, but were made in a rough field laboratory, hastily equipped in an old pumping station and storeroom, where there was more or less confusion resulting from the operation of large industrial plants in the immediate vicinity. Moreover, the chemical work was all done on a wholesale plan, under considerable pressure, so that no great attention could be paid to the details connected with any one operation. This was necessary in order to give capacity to the chemical laboratory, otherwise the large number of evaporations and titrations necessary could not have been executed.

On the other hand, it must be equally impressed on the mind that the best work done in this laboratory will undoubtedly rank with the best commercial laboratory practice. In fact, much that has been accomplished there will compare favorably with chemical research work of much greater pretension. It may be safely stated that the averages of half a dozen titrations as usually performed there will check the average of another similar set well within one-tenth of 1%, one-twentieth of 1% having been the error frequently observed. We have read much correspondence by eminent chemists on the subject of the accuracy of silver nitrate titrations for chlorine, and the accuracy obtained in our titrations will be better appreciated when it is stated that chemists as a rule do not rely on such titrations as being accurate to within 1 or 2 per cent. We have, however, to thank Dr. Mellet for his extended studies, which have resulted in a determination of the proper strengths of the solutions necessary to secure a degree of accuracy well within one-tenth of 1 per cent.

Nor is the degree of accuracy attained in the turbine tests to be inferred from the test titrations of Table 4, for most of these test titrations were crowded in at such times as the chemists chanced to be under less pressure of work than usual, and each series of test titrations, therefore, represents a break in the usual laboratory pro-

TABLE 4.—Errors of Titration Due to Unbalanced Evaporations.
Experimental Titrations in Chemical Laboratory.

(1)	(2)		(3)		(4)		(5)	(6)	(7)	(8)
Date, 1914.	Normal head-water.		Diluted salt solution.		Dosed tail-water.		N. H. + D. S. S. (Col. 2 + 3).	Diff. (Col. 5 — 4).	Error in percentage. (Col. 6 ÷ 4).	Remarks.
July 2 and 3	......	......	53.50	53.60	78.30	78.55				N. H. W. sample = 1 liter canal water.
	......	......	53.55	53.60	78.85	78.75				Salt solution sample = 10 c.c. of solution.
	......	......	53.70	53.55	78.65	78.65				Dosed tail-water sample = 1 liter of canal water + 10 c.c. of salt water solution.
	25.45	25.45	53.70	53.60	78.45	78.70				$AgNO_3$ solution = 1.56 grammes per liter.
	25.60	25.55	53.40	53.35	79.05	79.05				
	25.55	25.65	53.30	53.40	78.90	79.00				
Totals	76.60	76.65	321.15	321.10	472.20	472.70				
	153.25	......	642.25	......	944.90	......				
Ave.	25.542	......	53.521	......	78.742	......	79.063	+ 0.321	+ 0.408	
Temp.	......	......	......	......	......	......				
July 6	25.75	25.70	53.90	54.05	78.95	79.00				N. H. W. sample = 1 liter of canal water.
	25.60	25.57	54.05	54.10	79.10	79.15				Salt solution sample = 10 c.c. of solution.
	25.63	25.63	53.80	54.00	79.30	79.40				Dosed tail-water sample = 1 liter of canal water + 10 c c. salt solution.
	25.75	25.75	53.95	53.95	79.05	79.10				$AgNO_3$ solution = 1.56 grammes per liter.
	25.60	25.50	54.00	54.00	79.30	79.10				
	25.70	25.55	54.10	54.05	79.30	79.33				
Totals	154.03	153.70	323.80	324.15	475.00	475.08				
	307.73	......	647.95	......	950.08	......				
Ave.	25.644	......	53.996	......	79.173	......	79.640	+ 0.467	+ 0.59	
Temp.	20.5°C.	......	......	......	20.5°C.	......				
July 8	25.55	25.55	54.00	54.00	79.45	79.45				N. H. W. sample = 1 liter of canal water.
	25.50	25.55	54.00	53.95	79.40	79.40				Salt solution sample = 10 c.c. of solution.
	25.55	25.80	54.00	54.00	79.35	79.40				Dosed tail-water sample = 990 c.c. of canal water + 10 c.c. salt solution.
	25.80	25.70	54.10	53.95	79.55	79.43				Add 1% N. H. W. titration to D. T. W. titration = 1 000 c.c canal water + 10 c.c. S.S.
	25.70	25.80	53.90	53.87	79.25	79.35				$AgNO_3$ sol. = 1.56 grammes per liter.
	25.90	25.80	53.85	53.85	79.35	79.20				
Totals	154.00	154.20	323.85	323.62	476.35	476.23				
	308.20	......	647.47	......	952.58	......				
Ave.	25.6833	......	53.9558	......	79.3817	......				
Correction:					+ 0.2568					
Cor. Av.	25.6833	......	53.9558	......	79.6385	......	79.6391	+ 0.0006	+ 0.0	
Temp.	......	......	......	......	......	......				

TABLE 4.—(*Continued.*)

(1)	(2)		(3)		(4)		(5)	(6)	(7)	(8)
Date, 1914.	Normal head-water.		Diluted salt solution.		Dosed tail-water.		N. H. + D. S. S. (Col. 2 + 3).	Diff. (Col. 5 — 4).	Error in percentage. (Col 6 ÷ 4).	Remarks.
Averages:										
July 3	25.542	......	53.521	......	78.742	......	......	......	......	
" 6	25.644	......	53.996	......	79.173	......	......	......	......	
" 8	25.683	......	53.956	......	79.639	......	......	......	......	
Totals	76.869	......	161.473	......	237.554	......	......	......	......	
Ave.	25.623	......	53.824	......	79.185	......	79.447	+ 0.2629	+ 0.332	Average results.
Sample	*A*	*A*	*B*	*B*	*C*	*C*				Ratio of dilution of:
Sept. 23	55.51	55.40	27.73	27.59	13.95	13.85	......	......	......(1)	$\frac{\text{Sample } A}{\text{Sample } B} = \frac{1\,002.96}{500.58} = 2.0036$
	55.49	55.42	27.75	27.80	13.84	13.82	......	......	......(2)	$\frac{\text{Sample } B}{\text{Sample } C} = 2.0036$
	55.52	55.44	27.73	27.94	13.90	13.85	......	......	......(3)	$\frac{\text{Sample } A}{\text{Sample } C} = 2.00362$
Totals	166.52	166.26	83.21	83.33	41.69	41.52	......	......	......	Ratio of dilution takes flask and temperature correction into account.
	332.78	......	166.54	......	83.21	......	......	......	......	
Ave.	55.463	......	27.757	......	13.868	......	......	......	......	$AgNO_3$ soln. = 1.56 grammes per liter.
Corr. Av.	55.463	......	55.614	......	55.672	......	......	......	......	
Diff.	— 0.209	......	— 0.058	......	......	......	......	......	......	These are successive dilutions with distilled water beginning with Solution *A*.
Error %	— 0.375	......	— 0.104	......	......	......	Samples *A* and *B* referred to Sample *C*.			
Temp.	......	18.6°C.	......	18.6°C.	......	18.6°C.				
August 7	13.20	13.20	27.85	27.75	40.95	40.95	......	......	......	N. H. W. sample = 500 c.c. of canal water.
	13.15	13.20	27.85	27.80	40.90	40.90	......	......	......	
	13.25	13.15	27.85	27.64	40.62	40.71	......	......	......	Salt solution sample = 10 c.c. of solution.
	13.10	13.18	27.75	27.75	40.80	40.70	......	......	......	
	13.10	13.10	27.75	27.70	40.70	40.65	......	......	......	Dosed tail-water sample = 500 c.c. canal water + 10 c.c. salt solution.
	13.05	13.05	27.75	27.70	40.70	40.65	......	......	......	
	13.25	13.35	27.65	27.70	40.75	40.90	......	......	......	$AgNO_3$ solution = 1.56 grammes per liter.
	13.33	13.45	27.90	27.70	40.83	40.85	......	......	......	
Totals	105.43	105.68	222.35	221.74	326.25	326.31	......	......	......	
	211.11	......	444.09	......	652.56	......	......	......	......	
Ave.	13.194	......	27.756	......	40.785	......	40.950	0.165	0.405	
Temp.	25.5°C.	......	26.0°C.	......	25.5°C.	......	......	......	......	

TABLE 4.—(*Continued.*)

(1)	(2)		(3)		(4)		(5)	(6)	(7)	(8)
Date, 1914.	Normal head-water.		Diluted salt solution.		Dosed tail-water.		N. H. + D. S. S. (Col. 2 + 3).	Diff. (Col. 5 — 4).	Error in percentage (Col. 6 ÷ 4).	Remarks.
August 17 ...	12.90	13.05	27.70	27.70	40.60	40.60				N. H. W. sample = 500 c.c. of canal water. Salt solution sample = 10 c.c. of solution. Dosed tail-water sample = 500 c.c. canal water + 16 c.c. salt solution. $AgNO_3$ solution = 1.56 grammes per liter.
	13.15	13.05	27.70	27.70	40.95	40.60				
	13.30	12.85	27.70	27.70	40.60	40.60				
	13.08	13.30	27.75	27.70	40.65	40.65				
	13.08	13.13	27.70	27.87	40.80	40.60				
	13.10	13.00	27.75	27.70	40.68	40.50				
	13.30	12.88	27.55	27.52	40.65	40.65				
	12.88	13.31	27.70	27.72	40.65	40.65				
Totals......	104.79	104.57	221.55	221.61	325.58	324.85				
	209.36	......	443.16	......	650.43	......				
Ave............	13.085	......	27.697	......	40.652	......	40.782	0.130	0.320	
Temp..........	22.5°C.	......	22.5°C.	......	22.0°C.	......				
August 25 ...	13.30	12.98	55.63	55.55	40.53	40.31				N. H. W. sample = 500 c.c. of canal water. Salt solution sample = 10 c.c. of solution. Dosed tail-water sample = 500 c.c. canal water + 10 c.c. salt solution. N. H. W. flask holds 500.5 c.c. D. T. W. flask ÷ 2 holds 501.5 c.c. Therefore, add 0.2% N. H. W. titration to D. T. W. titration for true value. $AgNO_3$ solution = 1.56 grammes per liter.
	13.02	13.00	55.60	55.60	41.13	41.25				
	13.00	13.00	55.60	55.60	41.35	40.35				
	13.15	13.08	55.62	55.62	40.13	41.35				
	13.18	12.96	55.60	55.61	33.90	40.35				
	13.20	13.00	55.68	55.60	47.80	40.76				
Totals..........	78.85	78.02	333.73	333.58	244.84	244.37				
	156.87	......	667.31	......	489.21	......				
Ave............	13.073	......	27.805	......	40.768	......				
Correction:					+ 0.026					
Cor. Av......	13.073	......	27.805	......	40.794	......	40.878	+ 0.084	0.205	
Temp........	20.7°C.	......	21.5°C.	......	20.7°C.	......				
September 26 and 28......	12.85	12.83	55.29	55.15	40.30	40.40				N. H. W. sample = 1 liter canal water and split by using ½ liter flasks. Salt solution sample = 10 c.c. of solution. Dosed tail-water sample = 1 liter canal water + 10 c.c. salt solution, split into two halves by ½ liter flasks. Flasks were used so as to eliminate a correction. $AgNO_3$ solution = 1.56 grammes per liter.
	12.90	12.90	55.10	55.25	39.90	40.87				
	12.92	12.90	55.25	55.22	40.20	40.39				
	12.90	12.88	55.20	55.21	40.42	40.88				
Totals......	51.57	51.51	220.84	220.83	160.82	162.54				
	103.08	......	441.67	......	323.36	......				
Ave............	12.885	......	27.605	......	40.420	......	40.490	+ 0.070	+ 0.173	
Temp.....	15.3°C.	......	15.0°C.	......	15.3°C.	......				

TABLE 4.—(*Continued.*)

(1)	(2)		(3)		(4)		(5)	(6)	(7)	(8)
Date, 1914.	Normal head-water.		Diluted salt solution.		Dosed tail-water.		N. H. + D. S. S. (Col. 2 + 3).	Diff. (Col. 5 — 4).	Error in percentage. (Col. 6 ÷ 4).	Remarks.
Averages:										
August 9...	13.194	......	27.756	......	40.785	......	......	......	......	
" 17..	13.085	......	27.697	......	40.652	......	......	......	......	
" 25..	13.073	......	27.805	......	40.794	......	......	......	......	
September 26	12.885	......	27.605	......	40.420	......	......	......	......	
Totals......	52.237	......	110.863	......	162.651	......	......	......	......	
Ave............	13.059	......	27.716	......	40.663	......	40.775	+ 0.112	+ 0.276	Average results.
						Mean per liter......		+ 0.224	......	......
Sample........	*A*	*A*	*B*	*B*	*C*	*C*				Ratio of dilution of:
September 25	55.31	55.30	27.55	27.50	13.77	13.80	......	......	......(1)	$\frac{\text{Sample } A}{\text{Sample } B} = \frac{1\ 002.91}{500.55} = 2.0036$
	55.38	55.28	27.63	27.61	13.78	13.85	......	......	......(2)	$\frac{\text{Sample } B}{\text{Sample } C} = \frac{1\ 002.91}{500.55} = 2.0036$
	55.23	55.29	27.62	27.70	13.79	13.84	......	......	......(3)	$\frac{\text{Sample } A}{\text{Sample } C} = 2.0036^2$
Totals......	165.92	165.87	82.80	82.81	41.34	41.49	......	......	......	Ratio of dilution takes flask and temperature correction into account.
	331.79	......	165.61	......	82.83	......	......	......	......	
Ave	55.298	.....	27.602	......	13.805	......	......	......	......	Ag NO_3 soln. = 1.56 gramme per liter.
Cor. Av........	55.298	......	55.303	......	55.420	......	......	......	......	
Diff...........	— 0.122	......	— 0.117	......	......	......	......	......	NOTE:	These are successive dilutions with distilled water beginning with Solution *A*.
Error %.......	— 0.222	......	— 0.211	......	......	......	......	......		
Temp..........	16.3°C.	......	16.3°C.	......	16.3°C.	......	......	......		

cedure, and must be looked on as being conducted in a less systematic and efficient manner than the regular tests.

In studying the test titrations of Table 4 it will be best first to examine the work done in August, as that done in July was at a time when the chemists were less accustomed to their duties, and the facilities were not so well arranged as at a later time.

The tests of August 7th and 17th were arranged so that a sample of normal head-water consisted of a 500-c.c. flask filled to the graduation mark with this water. A sample of salt solution consisted of the quantity discharged from a 10-c.c. pipette, according to the directions set forth in the rules for the laboratory procedure, given in Section 88. A sample of dosed tail-water consisted of a 500-c.c. flask of normal head-water and a 10-c.c. pipette of salt solution mixed in a flask. It was scrupulously observed that the same flask and the same pipette were used throughout by any chemist in making up a set of three such samples. The salt solution was made of such a strength that the titration of this sample would run about half what it did in the usual test, and the normal head-water and dosed tail-water samples would run about as the usual ½-liter samples.

In the case of the tests of August 25th and 26th to 28th, a somewhat different procedure was adopted. The salt solution was of about double the strength of that used on August 7th and 17th, and a sample of dosed tail-water was composed of approximately the half part of a mixture of 10 c.c. of salt solution and a liter of normal head-water. As before, only one flask and one pipette were used by any chemist in making up a set of samples. The exact division of the mixture of salt solution and dosed tail-water into ½-liter samples, was not attempted, thus leaving apparent inconsistencies among the records of the titrations of dosed tail-water samples, for these tests. The sum of the titrations of mates, however, is quite consistent from pair to pair, considering that no extra pains were taken during the work. In fact, the work was not as good as that done in the regular tests, as has been explained. Thus, the titrations of the samples on August 25th and 26th to 28th were arranged to represent, as nearly as convenient, the actual titrations of a regular test.

As a consequence of the method of making up the samples and conducting the titrations, the sum of the normal head-water and salt

solution titrations, of any set, should, theoretically, be equal to the titration of the dosed tail-water sample of the same set. For example, on August 7th, we have 13.194 + 27.756 = 40.950, as compared with 40.785. Thus, the sum of the normal head-water and salt solution titrations exceeds that of the dosed tail-water by (40.950 — 40.785) = 0.165, an error of about 0.4% of the dosed tail-water titration.

This error is due to the fact that the concentration of the three samples is not the same. Indeed, the samples should be exactly alike as to all their characteristics, if the error of titration is to be eliminated. Dr. Mellet has accomplished the equation of the concentrations by proportioning sizes, dilutions, and evaporations in such a way that all samples will titrate alike. This, however, removes only a part of the difficulty, for the evaporations of the tail-water and normal head-water will necessarily be unbalanced, and the salt solution samples are dilutions with distilled water, instead of evaporations for concentration. As an example, it may be seen that it would have been necessary to evaporate about three times as much normal head-water as tail-water, in the tests now being discussed, in order to equate titrations, thus, perhaps, eliminating at least a part of the error detected above and observable in all the other tests.

Unfortunately, however, the equation of titrations requires unbalanced evaporations, and does not equate the errors of titration entirely, because there are other characteristics than concentration which must be equated simultaneously, as has already been pointed out in the theory which has been developed by the writer, in order to eliminate completely the errors of titration. There are organic and other substances in the normal head-water, which, when concentrated, operate in an entirely different manner to modify the titration of a sample of this solution as compared with a sample of either the dosed tail-water, brought to the same concentration, or the dilute salt solution sample, which contains no organic or other substances than the chemicals used in the test.

These differences are so marked that a little experience, even with such clear water as that of the St. Lawrence River, enables a person to know the class to which a particular sample belongs by mere inspection of its appearance. When evaporated in a casserole to a concentration of about 6 mg. in 10 c.c. of solution, the normal St. Lawrence

River water is reduced to a rusty muddy mixture, and the dosed tail-water sample, brought to the same concentration with only about one-quarter of the relative evaporation, is much cleaner and much more easily treated. The dilute salt solution of the same concentration, of course, contains no organic matter, and is, therefore, by far the clearest and best of all the samples.

This unbalancing of the evaporations, and the differences between concentrating by evaporation and diluting with distilled water modify to such an extent the ability to detect the end points of the three classes of titration that the errors of titration are not only still present, but may even be increased by completely equating the titrations. This remark applies to the method of concentrating the normal head-water sufficiently to equate the titrations, and the difficulty, it may be remarked, is the more serious where the normal head-water contains less chemical initially.

The differences between the tests of August 7th and 17th and the tests of August 25th and 26th to 28th have been pointed out. It is not surprising, therefore, to note that there is a very observable effect on the relative errors of the end points. Thus, in the tests where 10 c.c. of dilute salt solution have titrated to only 27 or 28 c.c., the errors are 0.165 and 0.130, respectively, though the titrations of the tests where 10 c.c. of salt solution have titrated to about 55 or 56 c.c. are only 0.084 and 0.070 c.c., respectively.

An exhaustive research for the causes of these discrepancies might be made, but it is apparent that the waters of different streams may differ, and, moreover, the waters of the same stream may differ at different times. It is therefore much better to eliminate this error by adopting the special methods which will be fully explained presently. On this account, no extended study of this particular error will be made except to compute the average correction to be applied to the titration of the normal head-water to reduce it to a value which is in error by the same percentage as that which affects the tail-water titration. Incidentally, the approximate correction to be applied to the titration of the dilute salt solution when it differs from the titration of the tail-water can be ascertained.

In order to make the determination, it is desirable to use all the reliable tests which have been made. This, however, will not include

the tests of July, as they are to be looked on as being more or less untrustworthy, notwithstanding the fact that the average corrections computed from them will not differ greatly from those computed by using the other tests. It will be possible, however, to use the dilution tests at the bottom of the table, and thus secure advantages from all the tests which may be considered as entirely trustworthy. Probabilities might be formally applied, but the following treatment will be quite sufficient, in view of the paucity of refined observations.

If we represent by c_1, c, and c_2 the average of the corresponding titrations for August 7th and 17th, and let x and y be the corrections to c_1 and c, respectively, to be applied to these quantities to reduce them to quantities having the same percentage of error as c_2, supposing that the errors are functions of the magnitudes of the quantities, we should have,

$$(c_1 + x) + (c + y) = c_2 \qquad (115)$$

In similar manner, for the tests of August 25th and 26th to 28th, observing that the normal head-water and tail-water samples titrate substantially the same as on August 7th and 17th, we should have for the average titrations

$$(c'_1 + x) + \frac{1}{2}(c' + z) = c'_2 \qquad (116)$$

For the mean titrations of the samples A and B of dilute salt solution at the bottom of Table 4, observing that, approximately, $A = c'$ and $B = c$, and that the ratio of dilution of A to get B is 2.0036 $= r$, say, we should have

$$\frac{1}{r}(A + z) = (B + y) \qquad (117)$$

Equations (115), (116), and (117) may now be written

$$\left.\begin{aligned} x + y &= c_2 - (c_1 + c) = -\,0.148 \\ x + \frac{1}{2}z &= c'_2 - (c'_1 + \frac{1}{2}c') = -\,0.090 \\ \frac{1}{r}z - y &= B - \frac{1}{r}A = +\,0.040 \end{aligned}\right\} \qquad (118)$$

which are easily computed from all the tests of Table 4, excepting those of July, which have been omitted for the reasons stated.

Thus:

$$\left.\begin{aligned} c_1 &= (13.194 + 13.085) \div 2 = 13.140 \\ c &= (27.756 + 27.697) \div 2 = 27.726 \\ c_2 &= (40.785 + 40.652) \div 2 = 40.718 \\ c'_1 &= (13.073 + 12.885) \div 2 = 12.979 \\ c' &= (55.610 + 55.210) \div 2 = 55.410 \\ c'_2 &= (40.768 + 40.420) \div 2 = 40.594 \\ A &= (55.298 + 55.463) \div 2 = 55.380 \\ B &= (27.602 + 27.757) \div 2 = 27.680 \end{aligned}\right\} \quad \ldots\ldots (119)$$

Apparently, we have three equations involving three unknowns, but it is observed that r is so nearly equal to 2 that small errors of observation introduce relatively large errors in the solutions. Thus, solving,

$$\left.\begin{aligned} x &= -\ 10.148 \\ y &= +\ 10.000 \\ z &= +\ 20.116 \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (120)$$

If r is placed equal to 2 on the left of the last of Equations (118), which may be done without appreciable error, and $\frac{1}{2} z$ is eliminated from the last two equations, we obtain

$$x + y = -\ 0.130$$

as compared with

$$x + y = -\ 0.148$$

in the first equation. Thus, we have two approximate values of $x + y$, and the most probable value is the arithmetical mean or,

$$x + y = -\ 0.139.$$

By assuming, now, that $z = -\ y$, which cannot be very far from the truth, we shall have, omitting the last of Equations (118),

$$\left.\begin{aligned} x + y &= -0.139 \\ x - \frac{1}{2} y &= -0.090 \\ \text{therefore} \qquad x &= -0.106 \\ \text{and} \qquad y &= -0.033 \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (121)$$

From the last of Equations (118) may be derived

$$z = -\ y = 0.027$$

a fair average value of y thus being about $-\ 0.03$.

The most important of these values is $x = -0.106$, which may be rounded off to -0.11, so that the correction per liter for titrations of 13 or 14 c.c. will be -0.22, and this figure must be deducted from the corresponding values of the doubled titrations, where the dosed tail-water titrates to about 40 c.c. Further, when the dilute salt solution titrates to about 55 and the tail-water to 40 c.c., the correction to be added to the titration of the salt solution is about 0.03 c.c. These values, of course, apply only to silver nitrate solutions of about the same concentrations as those used on the tests just discussed, which were about 1.5622 grammes per liter in all cases. The indicator, too, must be the same in all cases. However, the corrections would be only slightly different for concentrations of about 1.45 grammes per liter, so that no great changes need be made in the latter case.

TABLE 5.—SUMMARY OF AVERAGE CORRECTIONS FOR TITRATIONS WHERE $t_2 = 40$ C.C.

(APPLICABLE TO SUCH EQUATIONS AS (12), (14), (17), (19), (46), (53), AND (99).)

$AgNO_3$ = 1.5622 Grammes per Liter.

Indicator = 50 Grammes per Liter.

Titration, in cubic centimeters.	Corrections when tail-water titrates to about 40 c.c.
13.0	− 0.11
27.5	− 0.03
40.0	0.00
55.5	+ 0.03

From Table 5 we may compile, approximately, Table 6.

TABLE 6.—SUMMARY OF AVERAGE CORRECTIONS FOR TITRATIONS WHERE $t_2 = 55$ C.C.

(APPLICABLE TO SUCH EQUATIONS AS (12), (14), (17), (19), (46), (53), AND (99).)

$AgNO_3$ = 1.5622 Grammes per Liter.

Indicator = 50 Grammes per Liter.

Titration, in cubic centimeters.	Correction, $t_2 = 55$ c.c.
13.0	− 0.14
27.5	− 0.06
40.0	− 0.03
55.5	0.00

NOTE.—These corrections apply to ½-liter samples.

44.—Comparison of Theories.—Balanced and Unbalanced Evaporations.—Having shown how to correct the net error of titration in the case of the theory involved in Equations (12), (14), (17), (19), (46), (53), (98), and (99), it will be well to study a little more closely the theory involved in Equations (27), (28), (39), (44), (45), (56), (106), (107), and (108), with the view of eliminating this error entirely.

The main feature which has been added to the theory in these equations is the very evident fact that titrations of samples which are just alike in all respects are in error by the same percentage when the titrations are performed with the same silver nitrate solution in the same way.

Take, for example, Equations (110) and (111), Sections 42 and 43, where

$$Q = q \frac{v\, c_s\, e - k\, v_2\, c_s\, e}{v_2\, c_s\, e - v_1\, c_s\, e} \qquad (122)$$

supposing that all titrations have been reduced to the corresponding quantities of silver nitrate per liter of solution. We at once perceive that the new restriction imposes the requirement that e is not the same for any two titrations unless the two samples are exactly alike in all respects.

Thus, if t, t_1, and t_2 are the actual titrations of the sample volumes V, V_1, and V_2 which have been reduced from the original measured volumes of the samples by dilution or evaporation, and if ρ is the ratio of dilution of the salt solution and ρ_1 and ρ_2, respectively, the ratios of the volumes of original measured sample to the volumes of the residues after evaporation, which we may term "ratios of evaporation", then we may write Equation (122) as follows:

$$Q = q \frac{\frac{\rho\, t}{V}\, c_s\, e - k\, \frac{t_2}{\rho_2\, V_2}\, c_s\, e_2}{\frac{t_2}{\rho_2\, V_2}\, c_s\, e_2 - \frac{t_1}{\rho_1\, V_1}\, c_s\, e_1} \qquad (123)$$

in which the ratio of chemical equivalents is now considered to be different for different samples unless the actual samples of volumes V, V_1, and V_2 are exactly alike in all respects.

Now, it is easy enough to impose the condition that $t = t_1 = t_2$, by simply co-ordinating the values of ρ, ρ_1, ρ_2, V, V_1, and V_2, thus

$$\left.\begin{aligned} c &= \rho \frac{t}{V} c_s\, e \\ c_1 &= \frac{t_1}{\rho_1\, V_1} c_s\, e_1 \\ c_2 &= \frac{t_2}{\rho_2\, V_2} c_s\, e_2 \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (124)$$

from which, for $t = t_1 = t_2$, supposing that c, c_1, and c_2 are known,

$$\frac{c\, V}{\rho\, e} = \frac{c_1\, \rho_1\, V_1}{e_1} = \frac{c_2\, \rho_2\, V_2}{e_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (125)$$

and, for the moment, supposing that $e = e_1 = e_2$ for equal titrations, and that S_1 and S_2 are the measured volumes which are respectively evaporated to V_1 and V_2,

$$\left.\begin{aligned} \frac{c}{c_1} &= \rho\, \rho_1 \frac{V_1}{V} = \rho \frac{S_1}{V} \\ \frac{c}{c_2} &= \rho\, \rho_2 \frac{V_2}{V} = \rho \frac{S_2}{V} \\ \frac{c_1}{c_2} &= \frac{\rho_2}{\rho_1} \frac{V_2}{V_1} = \frac{S_2}{S_1} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (126)$$

Thus, having decided on the value of ρ, the relative sizes of V, S_1, and S_2 become known, and the absolute values are known, if, in addition, the value of V is chosen. Now, as c_1 and c_2 cannot be equal, and as the actual volumes titrated must be the same, if the samples are to titrate equally and be of the same volume, we must have, for the moment, by our less restricted theory,

$$V = V_1 = V_2 \ldots\ldots\ldots\ldots\ldots\ldots (127)$$

if we are to have $e = e_1 = e_2$. Therefore

$$\frac{\rho_2}{\rho_1} = \frac{c_1}{c_2} \ldots\ldots\ldots\ldots\ldots\ldots (128)$$

which shows that the samples of normal head-water and tail-water must be concentrated by evaporation by very different quantities. Thus, if $c_1 = 10$ mg. per liter and $c_2 = 40$ mg. per liter, it is clear that we

shall have to evaporate four times as much normal head-water as tail-water to have equal titrations of equal volumes.

By our restricted theory, however, the samples of head-water and tail-water will not, by such a process, be alike in all respects, and therefore we shall have to deny that equal titrations of samples of equal volumes imply that $e = e_1 = e_2$. Hence, equations of the type of (14), (17), (46), (53), (98), (99), etc., do not admit of an elimination of the net error of titration, for the reasons just made apparent, except by making an independent study of the net corrections to be applied directly to titrations similar to that which has been previously made, Tables 5 and 6, Section 43.

45.—Elimination of the Error of Titration.—The error of titration being principally due to the dissimilarity of the conditions under which the head-water blank and the dosed tail-water samples are titrated, it is clear that this source of error may be avoided by eliminating the head-water titration and substituting for it the special dilution titration, which requires the use of such equations as (27), (28), (39), (44), (45), (56), (106), (107), and (108), and the preparation in the laboratory of a special sample as nearly like the tail-water sample as possible.

In order to have the "special dilution", as it will be called, exactly like the tail-water sample, it must be made from the same salt solution which has been used in the test; and the ratio of dilution of this salt solution with normal head-water must be as nearly as possible the same as that in the test. Moreover, the evaporation of these special samples must be conducted in exactly the same manner as the evaporation of tail-water samples, and to the same extent. This can easily be done by measuring out equal quantities of each and evaporating them over a water bath to the same extent.

It has already been explained, in Section 12, that it will always be possible to have sufficiently accurate advance knowledge of the value of the actual ratio of dilution, *R*, in the test, to enable a special dilution to be made in the laboratory substantially like the tail-water sample. It has also been explained, Section 17, that we need not measure accurately the ratio of evaporation of samples, merely making them alike, nominally, as small differences of concentration due to small differences in evaporation will not affect the titrations perceptibly.

Hence, this procedure is equivalent to the formal statements of the equations

$$\left.\begin{aligned} \rho_2 &= \rho'_2 \\ V &= V_2 = V'_2 \\ c &= \rho \frac{t}{V} c_s\, e \\ c_2 &= \frac{t_2}{\rho_2 V} c_s\, e_2 \\ c'_2 &= \frac{t_2}{\rho_2 V} c_s\, e'_2 \end{aligned}\right\} \qquad (129)$$

It will be sufficient to discuss the nature of this procedure as it affects any one of the equations previously enumerated, for example, the first of Equations (106), Section 35, which is

$$r = \frac{r'}{1 + R' \dfrac{c_2 - c'_2}{c - k''\, c_2}} \qquad (130)$$

Substituting from Equation (129), and reducing somewhat,

$$r = \frac{r'}{1 + R' \dfrac{t_2 - t'_2}{\rho\, \rho_2\, t \dfrac{e}{e_2} - k''\, t_2}} \qquad (131)$$

since, by our special laboratory process, e_2 and e'_2 are substantially equal.

Now,

$$T = \rho \rho_2\, t \qquad (132)$$

is the titration of a sample of salt solution of the same size as the measured tail-water, or special dilution, samples, as computed from the actual titration of a salt solution sample of the same volume as the evaporated samples of tail-water or special dilution. That is, ρt is the titration of a sample of salt solution of the size of the evaporated samples, and ρ_2 is the ratio of the size of the measured sample to the size of the evaporated sample.

Thus, R' approximating R, we have,

$$r = \frac{r'}{1 + R' \dfrac{t_2 - t'_2}{T \dfrac{e}{e_2} - k''\, t_2}} \qquad (133)$$

in which T is the computed titration of a salt-solution sample of the same volume as that measured for the tail-water and special dilution samples based on the titration of a dilute salt-solution sample of the same size as the nominal volume of the evaporated samples.

It may be readily seen that, when t_2 and t'_2 are nearly equal, an error in T has no appreciable effect on the value of r, and that when $t_2 = t'_2$, the result is entirely independent of T, and therefore of any error in T.

It may be observed that e and e_2 are not necessarily equal, as the sample of dilute salt solution, though of exactly the same size as the others, is different in other respects. It is a dilution with distilled water and, therefore, does not contain the same constituents as the others. This will, in a small degree, affect the end point determination; but it must be observed that the error is relatively small, under the conditions imposed, and affects only those terms of the equations which are themselves relatively small and tend to vanish altogether when the conditions of Equations (129) and (133) are approached. Therefore, it may be concluded that the special laboratory process outlined in the foregoing theory may be used to obviate the necessity for correcting titrations, as the chemical procedure involved eliminates the value of the concentration of the normal head-water from the equations, and with it the necessity of dealing directly with any of the properties of the head-water.

That the magnitude of any residual systematic error will not affect the result of a test appreciably, may be ascertained by discussions similar to those of Sections 16 and 43, where it is shown that the systematic error due to titrations of dissimilar samples in the turbine tests did not affect the results given by the method of special dilutions by perceptible quantities.

As Equations (106) and (107) express the same conditions, the error will be exactly the same in the two cases. Indeed, Equations (106) may be derived from Equations (107) by eliminating c and solving for r, or R, as the case may be, although it is a solution where k'' has been inserted for a function of k', k, r', and r, which has a value nearly equal to unity. See Equation (96), Section 32.

Independently of Equations (106), it may be shown that an error in c tends to be eliminated in Equations (107). Suppose, for example,

that c becomes $c + \delta$ by reason of the error, δ. Then the resulting error in c_1 is

$$\frac{R' c_2 - (c + \delta)}{r'} - \frac{R' c_2 - c}{r'} = - \frac{\delta}{r'} \qquad (134)$$

and the corresponding error in r is

$$\frac{(c + \delta) - k c_2}{c_2 - \left(c_1 - \frac{\delta}{r'} \right)} - \frac{c - k c_2}{c_2 - c_1} \qquad (135)$$

which reduces to

$$\frac{\delta - \frac{\delta}{r'} \frac{c - k c_2}{c_2 - c_1}}{(c_2 - c_1) + \frac{\delta}{r'}}. \qquad (136)$$

When r and r', however, are closely equal, as is the case supposed in the use of Equations (107), we must have, closely,

$$r' = \frac{c - k c_2}{c_2 - c_1} \qquad (137)$$

so that the error in r, expressed by Equation (136), tends to vanish when r and r' are closely equal.

46.—Errors Considered Separately.—Error 1.—Initial Content of Chemical.—It is clear that in eliminating c_1 by Equations (107), or similar equations, as mentioned in Section 37, and the special dilutions, the properties of the normal head-water are eliminated. This is the first source of error enumerated in Section 41. Equations (27), (28), (39), (44), (45), (56), (106), (107), and (108), result from the elimination according to the methods to be used in the test. By using any of these equations in connection with special dilutions, the error resulting from this source will be entirely eliminated, and, along with it, the necessity for titrating the blank sample of head-water. Indeed, it is possible to eliminate this error even where the initial content is variable, as will appear in the next section. In order to gain some idea of the nature of the normal head-water, as regards initial content of chemical, a study of Table 7, giving the values, in cubic centimeters, of silver nitrate during the entire series of tests, will be enlightening.

It will be observed from the figures in Table 7 that the content of chlorine in the St. Lawrence River was almost, if not quite, constant during the tests. The increase of the titration from about 26

c.c. of silver nitrate to 28 c.c. near the close of the tests is due to a change in the strength of the nitrate from a concentration of 1.5622 grammes per liter to 1.45 grammes dissolved in 1 liter of distilled water, which would correspond to a concentration of about 1.449 grammes per liter.

TABLE 7.—Initial Content of Chlorine in Normal Head-Water During the Turbine Tests, Expressed in Hundredths of Cubic Centimeters of Silver Nitrate Solution per Liter of Sample, when the Titration is of ¼ Liter Evaporated to 10 c.c. in a Casserole of about 11 cm. Diameter. The Strength of the Silver Nitrate was Approximately 1.56 Grammes per Liter in all the Tests Except in the Second Series on Unit No. 11 when it was about 1.45 Grammes per Liter. The Strength of the Chromate of Potassium Indicator Was about 50 Grammes per Liter in All Cases, and the Quantity Introduced into the Sample was from Two to Three Drops by a "TK" Dropping Bottle. The General Rules for the Titrations Are Given in Sections 88 and 89. The Figures Are Corrected to Agree with the Second Part of This Table, See Section 43, Tables 5 and 6.

(Note: A small error in titration makes a relatively large error in the tabulated values herein.)

Unit No. 11.		Unit No. 12.					
Test.	V_1, in $\frac{1}{100}$ c.c.	Test.	V_1, in $\frac{1}{100}$ c.c.	Test.	V_1, in $\frac{1}{100}$ c.c.	Test.	V_1, in $\frac{1}{100}$ c.c.
105	2 595	200	2 564	205	2 463	210	2 479
106	2 574	201	2 559	206	2 483	211	2 479
107	2 585	202	2 545	207	2 474	212	2 480
108	2 561	203	2 525	208	2 485		
109	2 590	204	2 475	209	2 484		
Unit No 13.							
G	2 502	*W*	2 516	16	2 573	32–*A*	2 573
H	2 504	6	2 565	17	2 574	32–*B*	2 575
I	2 504	7	2 579	18	2 573	33	2 556
K	2 512	8	2 494	19	2 506	39	2 588
L	2 506	9	2 582	20	2 591	40	2 606
N	2 474	10	2 608	21	2 536	41	2 607
O	2 506	11	2 606	22–*A*	2 595	42	2 603
S	2 535	12	2 555	22–*B*	2 642	43	2 594
T	2 570	13	2 629	23–*A*	2 565	44	2 566
U	2 571	14	2 626	23–*B*	2 702	45–*A*	2 562
V	2 570	15	2 555	26–*A*	2 569	45–*B*	2 647
				26–*B*	2 616	46	2 584

TABLE 7 (*Continued*).—Initial Content of Chlorine in Normal Head-Water During the Turbine Tests, Expressed in Hundredths of Cubic Centimeters of Silver Nitrate Solution per Liter of Sample, Computed from the Titrations of Special Dilutions and Salt Solutions. The Strength of the Nitrate and Chromate of Potassium Indicator Was the Same in All Cases as Those Indicated in the First Part of This Table.

Computing Values of V_1 from Special Dilutions.

Unit No. 11.

Test.	V_1, in $\frac{1}{100}$ c.c.	Test.	V_1, in $\frac{1}{100}$ c.c.	Test.	V_1, in $\frac{1}{100}$ c.c.	Test.	V_1, in $\frac{1}{100}$ c.c.
105	2 560	111	2 747	116	2 775	121-*A*	2 813
106	2 575	112	2 823	117	2 719	122-*A*	2 751
107	2 564	113	2 775	118	2 816	*120-*B*	3 116
108	2 618	114	2 822	119	2 802	121-*B*	2 793
109	2 637	115	2 782	120-*A*	2 825	122-*B*	2 807

Unit No. 12.

200	2 534	204	2 477	208	2 512	212	2 460
201	2 561	205	2 467	209	2 474		
202	2 605	206	2 470	210	2 458		
203	2 509	207	2 459	211	2 469		

Unit No. 13.

23-*B*	2 550
26-*B*	2 749

Thus the tests on Unit No. 11 in September—Nos. 105-109—average 25.92, and those in November—Nos. 111-122—average 27.98. In the former case the concentration of the nitrate was 1.5622; in the latter case it was about 1.449. Hence the titration in November, computed from that in September, should be

$$25.92 \times \frac{1.5622}{1.449} = 27.95$$

which agrees very well indeed with the average for November mentioned previously.

The concentration of nitrate in November was taken at about 1.45 grammes per liter so as to facilitate the determination of the approximate salt content of the normal head-water from the titration of a sample by simply dividing by 2. Concentrations computed in this

way, however, are subject to the error of titration. This error, for normal head-water, is in the neighborhood of 1% on the side of excess.

47.—Error 2.—Variability of Initial Content.—Whether the elimination of the error due to initial content is complete or only partial depends in large measure on whether this content is variable and on the method of taking samples.

In order to eliminate the error due to variability of content, it is necessary only to secure an average sample. Such an average must be a volume average and not a time average. However, as the velocity at any point in the cross-section at the head-water sampling station is nearly constant, a continuous sample taken at a uniform rate during any test, or part of a test, will represent a fair average for that point during the observation. If the variability is so great as to cause very different concentrations in different parts of the section at the same time, more than one sample will be required for the normal head-water. If, in addition, there is much variation in velocity from point to point of the cross-section, the concentrations of the samples from these points would have to be weighted in proportion to the relative velocities at the same points.

Fortunately, the latter precaution is seldom needed; a single head-water pump, operating continuously during a test, will suffice in the majority of cases. Indeed, the constancy of salt content can be checked by taking several continuous samples from each pump during a test, and noting that the special dilutions resulting from these samples all titrate substantially alike.

Table 7, Section 46, gives a list of normal head-water titrations observed during the tests.

48.—Error 3.—Variability of Standard Chemical.—If the concentration of the silver nitrate should vary during a series of titrations, it may introduce an error of any magnitude, depending on the extent and character of the variations. At first sight, it would appear that this error cannot be eliminated; but, on closer study, it will be apparent that if the methods advocated in this paper are followed, gradual changes may be eliminated by group titrations. Suppose, for example, that two chemists are each titrating the same test in two independent treatments. It will not matter if their silver nitrate solutions are different; they should arrive at the same result, notwithstanding, for titrations vary in magnitude in an inverse manner

with variations of concentration, being nearly in an inverse ratio thereto, and, as the methods advocated herein require that all titrations shall be approximately equal, they will be affected in any test by the same constant error, which, therefore, will cancel from the equations, because only the ratios of the titrations enter such equations.

If, therefore, the same number of duplicate samples of each solution are prepared, the samples may be titrated in groups containing one or more samples of each solution. Only one chemist should titrate any one group, and, during its titration, he should be sure to use the same instruments, the same silver nitrate solution, and the same indicator solution. He should even go so far as to approximate the same readings of his burette and other scales, in measuring his solutions. In fact, the burette should be filled nearly, if not exactly, to the zero of its scale before each titration, and should be proportioned to the work so that it will be almost emptied by each titration, with only a good practical margin to spare. This is on the side of accuracy and economy. The reason for this is more apparent when it is observed that, by following these directions, each titration of any group will be affected with the same constant error, including the personal error of the chemist, as only one chemist is permitted to titrate any one group of samples.

Not only should the samples be thus divided into groups, but each group must contain nearly the same number of samples of each kind, so that they may be titrated one kind after the other as rapidly as possible, with perhaps an extra sample of one kind, so that the titration of the group may begin and end with that same kind of sample.

For example, suppose a test has been run in which there are twelve tail-water samples, T_1, T_2, etc., four special dilutions, D_1, D_2, etc., and four salt solution samples, S_1, S_2, etc. Then, to eliminate the errors due to changes in the nitrate, the personal error of the chemist, and errors of the graduation marks on the burette, the preceding directions must be followed, and the groups may be arranged as follows:

GROUP TITRATIONS.

Group 1..........	T_1	D_1	T_2	S_1	T_3
Group 2..........	T_4	D_2	T_5	S_2	T_6
Group 3..........	T_7	D_3	T_8	S_3	T_9
Group 4..........	T_{10}	D_4	T_{11}	S_4	T_{12}

A SECOND GROUPING.

T_1	D_1	T_2	S_1	T_3
T_4	S_2	T_5	D_2	T_6
T_7	D_3	T_8	S_3	T_9
T_{10}	S_4	T_{11}	D_4	T_{12}

There appears to be no good reason for the second grouping, rather than the first, though some chemists may prefer it.

The second grouping might cause confusion in entering the record on the record sheets, with the possibility of making an error by entering a special dilution in a salt solution space. This error, however, would be small if the directions are followed closely, as all titrations of the same group are nearly equal among themselves.

Each group should be titrated as rapidly as possible, by the same chemist, as previously explained, and in the order of reading from left to right, or the contrary. As a consequence of this procedure, it will be seen that gradual changes in the chemicals used, such as increasing concentration due to evaporation or changes in temperature, will not affect the results if the work is executed as outlined.

49.—Errors 4 and 5.—Indicator, and Excess of Standard.—The determination of the end point of any reaction is always affected by an error. Part of this error may be due to reactions between the indicator and standard solutions used and part may be due to the necessity of having an excess of standard in order to make distinct the indication of the end point. In either case, the titration, as determining the quantity of chlorine, or other chemical, is in error.

So far as determining the quantity of water, or other liquid, is concerned, however, it will not be in error if the method of special dilutions developed in the tests described herein is used. For such an excess of titration will affect all the titrations alike, as they are of nearly the same magnitude for substantially equal samples.

Indeed, it follows as a consequence of the methods laid down that constant errors of titration are such only as relates to the exact determination of the quantity of chemicals present. They are not errors at all as relates to the determination of volumes of liquids by the method of special dilutions.

50.—Error 6.—Uniformity of Mixture and Distribution of Salt in Tail-Water.—It seems that many engineers have an idea that failure to secure uniformity of mixture means failure to secure an

accurate test. That such is not the case may be readily seen from the following and similar considerations.

In the tests described there were two draft-tubes delivering the discharge from a pair of vertical twin runners. In some of the tests the distribution of salt between the upper and lower draft-tubes was nearly equal; in other cases there was as much as 15% and even 20% more in one draft-tube than in the other, as may be seen by examining the titrations of the tail-water samples which are collected at the end of this paper. Let it be required to determine the possible errors due to this cause:

It is clear that it is a volume average of concentration which is required for the value of c_2 in the equations. Therefore, if the concentration varies from point to point in the cross-section of the tail-race, the arithmetical mean of the concentrations of several samples taken at these points will not be an average concentration unless the velocity is uniform at all points, which is never the case. It is necessary, therefore, to determine the weighted mean, the weight of any concentration being the ratio of the quantity of water discharged at that concentration to the total quantity discharged during the test. Thus, if W_1 is the volume of water discharged by the upper draft-tube at the average concentration, c_1, and W_2 is that discharged by the lower draft-tube at the average concentration, c_2, then the weighted mean, or volume-average of the concentration for the test, will be

$$M = c_1 - \frac{W_2}{Q}(c_1 - c_2) \ldots\ldots\ldots\ldots\ldots\ldots (138)$$

where Q is the total discharge of both draft-tubes during the test.

Take, now, the most unfavorable case, where the concentrations in the upper and lower draft-tubes are, respectively, 55 and 45 c.c. of silver nitrate. Further, suppose that the upper draft-tube discharges 20% more water than the lower one. This, of course, is a far greater difference than can reasonably be imagined, as the runners are exactly alike. However, with this excessively unfavorable assumption, observing that the discrepancy between the discharges is about 10% of the total discharge, we have,

$$M = 55 - \frac{50}{110}(55 - 45) = 50.45$$

as against

$$\frac{55 + 45}{2} = 50$$

for the arithmetical mean. Thus, the error introduced is less than 1% under assumptions which must be more unfavorable by far than the actual conditions under which the test was executed. It would be much more like the actual conditions to assume that the upper runner differed from the lower by only about 5%, in which case the weighted mean would be

$$M = 55 - \frac{50}{102.5} \times 10 = 50.12$$

which does not differ from the arithmetical mean by as much as $\frac{1}{4}$ of 1 per cent. This is probably too large an error, as the gates of the two water-wheels are undoubtedly set so as to cause a nearer approach to equality of discharge than that last supposed.

It follows, from the preceding comparisons of the arithmetical and weighted means, that, if we have anything approaching the true weights of the concentrations observed in the tail-race, we can compute an average concentration which will differ from the true average by an error which is measured by only a few hundredths of 1%, and this, too, where there are considerable variations among the observed concentrations and velocities.

The foregoing remarks apply particularly to the case of two draft-tubes. In the case of a single draft-tube, or tail-race, there are likely to be greater variations of relative velocity from point to point than has been supposed to exist between the two draft-tubes. Thus, this case requires a study of the errors similar to that for the case of a pair of draft-tubes, but it will usually be found that the distribution of salt over the section of a single draft-tube is so much better than it is between two draft-tubes that the resulting error is even less than those just discussed.

Take, for example, one of the best tests on Unit No. 13, such as Test 41. The detailed distribution of concentration in the draft-tubes is shown by Table 8.

TABLE 8.—*DISTRIBUTION OF CONCENTRATION IN TEST 41, AS SHOWN BY THE TITRATION OF THE EIGHTEEN TAIL-RACE SAMPLES.

	West Half.			East Half.		
Upper draft-tube.	53.20		53.92		54.40	
		54.45		53.85		54.94
	53.45		54.70		54.85	
Lower draft-tube..........		53.85		54.05		53.25
	54.05		53.80		54.30	
		53.90		54.20		54.55

* The units are cubic centimeters of silver nitrate consumed in titration.

It will be seen at once that the lowest concentration is 53.20, and the highest is 54.94, or a maximum variation among samples of only a trifle more than 3% of the average. It follows that the arithmetical mean, 54.095, is certainly within 1½% of the truth, and the probable error is only a small fraction of 1%, the assumption being that the average distribution of chlorine among the various samples is uniform, and that the figures of Table 8 vary by accidental errors due to one cause or another.

If, on the other hand, it is considered that the variations in Table 8 are real variations of concentration, then we must seek to weight each concentration by the relative velocity at the point where it is observed. It will be better engineering, however, and far more economical of time, where a number of tests are to be thus discussed, to divide the upper and lower draft-tubes into east and west halves, thereby dividing the tail-race into quarters, and then to weight the mean concentration of each quarter with the mean relative velocity of that quarter. It may be remarked, also, that we are now regarding the tail-race as the single discharge passage from a turbine unit. This is reasonable, as the concentrations are so nearly uniform that there is no material variation of the mean concentrations between the upper and lower halves of the tail-race. Of course, attention might be confined to either upper or lower draft-tube, if one wished to be very consistent. The result will be the same in either case.

The mean concentrations of the four quarters may then be arranged as follows:

Comparison by Quarters.

53.944	54.510
53.900	54.070

By thus taking means for the quarters, the maximum variation among the four averages for the samples of each quarter has been diminished to 0.61 c.c., or only a trifle more than 1%, as compared with 3% when the samples are taken separately. It may well be concluded, then, that the arithmetical mean of all samples is well within ½ of 1% of the truth, at least in this particular case, and, consequently, that the weighted mean will be within a small fraction of 1% of the truth.

It will also be of interest to compare the upper and lower and east and west halves of the tail-race concentrations, thus:

COMPARISON BY HALVES.

Mean of nine samples, upper half	54.196
lower half	53.994
Mean of nine samples, west half	53.922
east half	54.290

These results show that the variation of average samples has again been reduced, and we note that the variation for the upper and lower halves is only 0.2 c.c., and it is less than 0.4 c.c. for the east and west halves. Owing to this narrowing of the variation, it is apparent that the probability is very high, in this particular case, that the arithmetical mean is not in error by more than $\frac{1}{4}$ of 1% at most, and a mean derived from anything like the true weights cannot be in error by as much as $\frac{1}{10}$ of 1 per cent. All this may be proved, with large margins in favor of the less formal conclusions, by a simple application of the principles of probability, but would not add anything to the probability of the fact which we show, namely: that the variability of concentration is not such a serious matter as many have supposed, and, above all, that it is quite possible, in testing large-capacity turbines operating on low heads, to secure remarkably uniform distributions, even where there are two water-wheels operating on one shaft and discharging through two separate draft-tubes.

By referring to Fig. 37 the proper weights for the concentrations of the four quarters may be shown to be about as follows:

	West.	East.	Mean.
Upper	0.30	0.20	0.50
Lower	0.33	0.17	0.50

Therefore, the weighted mean for this test may be computed very closely as follows:

$$0.30 \times 53.944 = 16.183$$
$$0.33 \times 53.900 = 17.787$$
$$0.20 \times 54.510 = 10.902$$
$$0.17 \times 54.070 = 9.192$$

Weighted mean.. 54.064

A comparison of the weighted and arithmetical means may be made, thus:

Arithmetical mean = 54.095

Weighted mean = 54.064

which are seen to differ by less than $\frac{6}{100}$ of 1%, with the probability that the weighted mean is in error by even less than this, as it has been repeatedly shown in the chemical laboratory that the means from sets of six or eight equal samples will titrate within $\frac{5}{100}$ of 1% of each other.

This latter, of course, is an accidental error as distinguished from the systematic error due to erroneous weighting which has just been discussed. This distinction must be carefully made in criticising accurately made tests; and the residual error, resulting from these two classes, must be shown not to be greater than a certain quantity before any conclusions can be drawn as to the accuracy of the tests.

By analyzing the salt distribution in the tail-race in a manner similar to the foregoing, a close estimate of the degree of accuracy attained in the weighted mean for the tail-race concentration in any particular test can be made, and thus the effect on the final result of the test may be ascertained. It is readily seen, therefore, that, so far as mixture and distribution of salt are concerned, the best tests are well within the desired degree of accuracy of $\frac{1}{10}$ of 1 per cent.

Of course, it is assumed, in making this last statement, that the weights are not vastly incorrect, although there is a very wide margin of error permissible. For example, by taking the weights twice as great, relatively, on the west side of the race, as they are in the foregoing table of weights, we would have:

	West.	East.	Half Tube.
Upper..................	0.375	0.125	0.50
Lower..................	0.400	0.100	0.50

which leads to the following calculation of the weighted mean, on the assumption that the weights used in the computation of the mean of 54.064, above, are only half as great relatively on the west side as they should be:

$$
\begin{aligned}
0.375 \times 53.944 &= 20.229 \\
0.400 \times 53.900 &= 21.560 \\
0.125 \times 54.510 &= 6.814 \\
0.100 \times 54.070 &= 5.407 \\
\hline
\end{aligned}
$$

Weighted mean, based on assumed weights......... 54.010

Thus, if the weights used in obtaining the mean, 54.064, are only half as large on the west as they should be, there would be introduced an error in the weighted mean of only 0.1 of 1 per cent.

Now, Fig. 37 is based on current-meter observations in the tail-races of the units tested, and, though the current-meter work is to be regarded as only a rough field operation, without correcting the meters for errors due to turbulence, it is difficult to suppose that the errors in the relative velocities, east and west, can be more than 15 or 20 per cent. Hence, it is concluded that no serious error results in the best tests by reason of errors in weighting the concentrations of salt in the tail-race samples.

Finally, it may be stated that the ultimate determination of the efficiencies of the units justifies the last conclusions, not only when tests of the same unit are compared with one another, but also when the tests of Units Nos. 12 and 13 are compared. These two units being exactly alike, their efficiencies should be equal, with only such differences as may be due to slight differences in hydraulic conditions, adjustment, packing, lubrication, etc.

51.—Distributing Pipes and Centrifugal Pump.—As explained in the description of the testing plant, Part III, the salt solution was injected into the suction of an 8-in. centrifugal pump, which discharged into a header feeding twelve 2-in. pipes. Six of these 2-in. pipes passed down the stop-log slot on the west and six on the east side of the head-race. These pipes are numbered from 1 to 12, the odd numbers being west, the even numbers east, and progressing downward.

There were six horizontal 2-in. pipes extending across the head-race, each perforated with three series of holes, one series along the top element discharging vertically upward, one series along the bottom element discharging downward, and the third series along the up-stream element discharging against the current. A pair of differential pulleys—one such pulley being at each side of the head-race—facilitated the adjustment of these six horizontal distributing, or sprinkling, pipes, so that they divided the rectangular cross-section of the race into six equal horizontal strips, with a pipe in the horizontal median of each strip.

The twelve distributing pipes from the header were connected by 2-in. hose to the ends of the risers leading to the horizontal sprinkling pipes in pairs, so that Nos. 1 and 2 supplied the upper, or No. 1, sprinkler, with salt solution; Nos. 3 and 4 supplied the next lower, or No. 2, sprinkler, and so on downward.

In the tests on Unit No. 13 these horizontal sprinklers were open throughout their length, but in some of the later tests a block of wood, or plug, was introduced at the middle of the pipe in the center line of the head-race.

The relative discharge from each of the various parts of the distributing system was controlled by gate-valves and gauges connected to the distributing pipes, 4 or 5 ft. below the point where they branched from the header. At these points there was also provided a pet-cock on each pipe, so that samples of the mixtures being pumped into the different pipes could be secured.

During the course of a number of the regular tests, samples were taken from these cocks and titrated so as to obtain an estimate of the relative quantities of salt passing through each of the twelve pipes leading from the header.

In this way it was proved conclusively that the method of introducing the salt into the centrifugal pump effected a perfect mixture with the water being raised from the canal by the pump, so that the discharge was a salt solution of perfectly uniform strength in all the pipes at any given time. The experiments also enable one to calculate the discharge of the centrifugal pump, although no effort was made to observe the power consumed by the pump or to compute its efficiency.

The observed titrations, uncorrected for temperature effects and calibrations of glassware, for two such tests as have been described, are given in Table 9, with approximate calculations for the discharge of

TABLE 9.—Showing Uniformity of Mixture Discharged from the 8-Inch Centrifugal Pump in Test *V*, Unit No. 13, August 10th, 1914.

The figures are in cubic centimeters of silver nitrate consumed in titration up to the end point.

Horizontal pipe No.	West.	East.
1	25.75	26.15
2	25 95	25.85
3	25.85	25 80
4	25.85	25 95
5	25.75	25.75
6	25.80	25.70
Means	25.825	25.867

Average of all 25.846
Average, less 0.07 c.c., by Table 5, Section 43 .... 25.78

the pump in all the tests where distributing-pipe samples were taken. The practical equality of the titrations of the twelve samples taken from the distributing pipes is very evident.

The value of v_2 for these twelve titrations, therefore, is about 25.78. A sample consisted of 10 c.c. pipetted from a stock solution made up by taking 20 c.c. of the discharge from a given pipe and diluting to 500 c.c. with distilled water. Thus, by multiplying the observed titration by $100 \times 25 = 2\,500$, there results a number which is, theoretically, what the titration of 1 liter of the actual discharge would have been. The uncorrected concentration of the salt solution is 544 900 and the concentration of the normal head-water, as usual, is about 25.72. Therefore, the calculation of the discharge of the pump for Test V, with considerable approximation, is

$$v = 544\,900$$
$$v_1 = 25.72$$
$$v_2 = 2\,500 \times 25.78 = 64\,450$$
$$q = 0.24 \text{ (from the record),}$$

and

$$Q = 0.24 \frac{544\,900}{64\,450} = 8.46 \times 0.243 = 2.06 \text{ cu. ft. per sec.}$$
$$= 2.06 \times 60 \times 7\tfrac{1}{2} = 928 \text{ gal. per min.}$$

The elevation of the surface of the canal water was 198.6, and that of the center of the pump was about 217, so that the suction lift was 18.4 ft. The speed of the pump was about 600 rev. per min.

TABLE 10.—Mixture Discharged from 8-Inch Centrifugal Pump in Test 26, Unit No. 13, August 26th, 1914.

Horizontal pipe No.	West.	East.
1	28.09	28.12
2	28.15	28.07
3	28.13	28.15
4	28.10	28.07
5	28.10	28.10
6	28.20	28.25
Means	28.128	28.127

Average of all 28.13
Average, less 0.06 c.c. by Table 5, Section 43 .... 28.07

Computation with approximate data from test:

$v = 528\ 600$

$v_1 = 25.72$

$v_2 = 2\ 500 \times 28.07 = 70\ 180$

$q = 0.24$ (from record),

$$Q = 0.24 \frac{528\ 600}{70\ 180} = 0.242 \times 7.54 = 1.82 \text{ cu. ft. per sec.}$$

$$= 1.82 \times 60 \times 7.5 = 820 \text{ gal. per min.}$$

Elevation of surface of canal water = 195.8

Suction lift = 217 — 195.8 = 21.2 ft.

Thus the effect of the additional suction lift is apparent in the decreased discharge, the pump running at about the same speed as before.

Table 11 gives a summary of all the titration tests of the discharge of the centrifugal pump, with foot-notes explaining certain apparently anomalous results due to varying conditions of operation.

TABLE 11.—Centrifugal Pump Discharge During Various Tests.*

Data, uncorrected for temperature.†	Symbol.	Test Number.						
		D	*E*	*S*	*V*	26	43	118‡
Titration of salt solution, in cubic centimeters........	v	502 700	581 000	545 700	544 900	528 600	542 500	62 140
Titration of head-water, in cubic centimeters.........	v_1	25.23	25.21	25.56	25.72	25.72	25.94	28.05
Titration of pump discharge, in cubic centimeters	v_2	62 000	81 650	78 150	64 450	70 180	74 680	7 348
Ratio of dilution............	K	8.12	7.12	6.98	8.46	7.54	7.27	8.48
Rate of discharge of salt solution, in cubic feet per second	q	0.257	0.281	0 233	0.243	0.242	0.241	0.242
Suction lift, in feet..........	......	16.9	17.5	17.7	18.4	21.2	20.7	19.3
Discharge of pump, in gallons per minute...........	Q	936	901	735§	928	820	788	924‖

Note.—The effect of a change in suction lift is very apparent in these tests. The readings of gauges on distributing pipes can be found in Tables 50, 51, 52, and 53. No exact pressures are available.

* The discharge of this pump should be added to the turbine discharge determined by the current meters in tests where the pump was running, as the pump discharge was shunted around the meter section.

† Although the data are uncorrected for effects of temperature, they are sufficiently accurate for the purpose of this table.

‡ Test 118 was on Unit No. 11. All the other tests of this table were on Unit No. 13. Unit No. 13 was equipped with a direct-current generator, but No. 11 was equipped for alternating current.

§ In Test *S*, all six of the feeders to sprinkling pipes on west side of head-race shut down, which accounts for the low discharge of pump in this test.

‖ In Test 118 another motor was used, which ran the pump at speeds considerably greater than 725 rev. per min. Two of the sprinklers, however, were shut down.

The principal information to be gained from these tests is that a centrifugal pump appears to be an excellent mixer, and, consequently, is especially susceptible to the chemical method of turbine testing. The pump was an old one, and the runner was in poor condition, so that the efficiency was undoubtedly low. Neither was the speed anywhere near high enough in most of the tests, all of which facts, undoubtedly, added to the mixing qualities of the pump.

52.—Distribution Tests.—It was shown in Section 51 that the distribution of salt among the twelve distributing pipes was uniform. The results of several tests on Unit No. 13 to determine the distribution of salt in the tail-race arising from the separate discharge of each sprinkling pipe and also the results of tests to ascertain the time required after starting a test to establish steady conditions of distribution in the cross-section of the tail-race will be discussed in this section.

The former of these matters was the subject of investigation in Test *M*, July 25th, 1914. The salt-solution tank was filled with a strong solution, the usual samples of which titrated as follows:

SALT SOLUTION, TEST *M*.

	54.80	54.85
	54.75	54.75
	54.80	54.80
	54.80	54.80
	219.15	219.20
Mean......	54.79	

After this each of the six sprinkling pipes was opened in succession for about 5 min., but with scarcely sufficient time intervening to permit the establishment of steady conditions in the pipes and tail-race. The sample bottles at the tail-race sampling pumps were changed each time so that separate sets of samples were obtained for each pipe. There were thus obtained 84 tail-race samples, 14 representing the relative distribution of salt in the tail-race for each of the six sprinklers.

The numbers of the pipes progress downward, so that Pipe 1 was the uppermost and Pipe 6 the lowest in the head-race distributing system. The tail-race sample pipes were in the cross-section at the stop-log slot of Unit No. 13, just outside the power-house. Some time

afterward this section was abandoned on account of the difficulty of keeping the ¼-in. pipes in repair.

TABLE 12.—TESTS *M*.—DISTRIBUTION OF SALT IN TAIL-RACE.

TITRATIONS OF THE INDIVIDUAL TAIL-RACE SAMPLES.

(The unit is 0.1 c.c. of silver nitrate, and each number represents the titration of ½ liter.)

Number of Sprinkling Pipe Open, Tests *M*.											
1.		2.		3.		4.		5.		6.	
W.	E.	W.	E.	W.	E.	W.	E.	W.	E.	W.	E.
322	295	306	285	230	292	170	210	138	144	136	138
378	270	293	288	272	270	169	200	136	144	137	142
337	269	287	269	212	264	166	206	120	152	138	156
152	167	160	180	201	231	259	258	300	255	282	283
137	137	147	153	192	212	263	267	318	300	322	321
136	135	146	150	202	204	268	272	317	208	313	329
267		315		266		183		138		144	
131		145		200		251		...		347	

NOTE.—The figures below the twelve regular samples represent, respectively, the relative concentrations at the center lines of the upper and lower draft-tubes, and were determined from samples taken from Pumps 41 and 42.

Table 12 shows the actual titrations of samples in the tests. These should be reduced by 13.5 c.c., that is, by 135 tenths, the titration of ½ liter of normal head-water, in order to secure numbers representing the relative salt content of the tail-water at the various sampling pipes resulting from the introduction of salt solution through the particular sprinkling pipe which is open in any particular test. It will make the different tests more directly comparable with each other, however, to reduce these resulting relative concentrations to percentages of the mean concentration for the whole tail-race by dividing each by this mean. The result of this compilation is given in Table 13, which, therefore, represents quite accurately the relative distribution of concentration in the tail-race due to each of the six sprinkling pipes.

From Table 13 it is apparent that Pipes 1 and 2 deliver nearly all their salt supply to the upper draft-tube, and Pipes 5 and 6 supply principally the lower draft-tube. Pipes 3 and 4 divide their supplies much more evenly, but Pipe 3, being above the middle of the cross-section in the head-race, naturally supplies a greater proportion to the upper draft-tube, and Pipe 4, being below the middle supplies a

greater proportion to the lower draft-tube. All this might easily have been anticipated, but Table 13 is very valuable, as it enables one to secure a uniform distribution in the tail-race by adjusting the valve on the proper pipe in the head-race. It was by using this table that good distributions were secured on the tests of Unit No. 13.

TABLE 13.—TESTS *M*.—SEPARATE DISTRIBUTIONS OF SALT IN TAIL-RACE OF UNIT NO. 13 DUE TO EACH OF THE SIX SPRINKLING PIPES IN THE HEAD-RACE BASED ON TWELVE SAMPLES OF TAIL-WATER.

(The figures represent percentages of the mean values of the concentrations at the twelve sampling pipes.)

NUMBER OF SPRINKLING PIPE OPEN, TESTS *M*.											
1.		2.		3.		4.		5.		6.	
W.	E.	W.	E.	W.	E.	W.	E.	W.	E.	W.	E.
201	172	196	172	98	162	39	82	3	10	2	3
262	146	182	175	142	139	38	71	2	11	2	8
217	144	175	154	80	133	34	79	—18	20	3	23
18	34	29	52	68	100	137	135	196	142	164	165
2	2	14	21	59	80	141	146	217	196	209	207
½	0	13	17	69	71	146	151	216	205	199	216

Table 12 gives additional evidence in favor of the conclusions of Section 50, that variations of distribution are not vitally important where the water in the tail-race is all flowing outward, that is, out of the tail-race. This is made clear by Table 14, a compilation of the average values of that part of the tail-race concentration in Tests *M* which is due to the separate sprinkling pipes. Thus, there were six tests compared in Table 12; taking the arithmetical average in each

TABLE 14.

Number of sprinkling pipe open.	Salt rate, in liters per second.	Average equivalent titration, in tenths of cubic centimeters.	Ratio of titration to salt rate.
1.	1.53	93	61
2.	1.63	87	53
3.	1.62	97	60
4.	1.61	90½	57
5.	1.71	84½	49
6.	1.78	90	50
Means	1.65	90.3	55

case and deducting 13.5 c.c., that is, 135 tenths, Table 14—a table of averages—has been compiled.

The salt rate is simply the rate of discharge of the salt solution during the times the samples were being taken in the various tests (about 5 min. in each test). The strength of the salt solution is stated in tabular form at the beginning of the section. A close agreement between the salt rate and the mean concentration in the tail-race need not be expected, nor required, in these tests, as the period of the tests was too short for such a degree of accuracy, and it is doubtful whether sufficient time was allowed between tests. Moreover, it does not appear that the salt rate was maintained anywhere near constant during any test, so that it is easy enough to account for these variations among the mean concentrations and the ratios between them and their corresponding salt rates. These facts do not detract in the least from the importance of the conclusion which may be drawn from the fact that the average equivalent titration per liter of salt rate varies by only 10 or 12% from the mean, 55, as a consequence of opening successively the six sprinkling pipes in the head-race, thereby varying the relative intensities of concentration in the various parts of the tail-race due to the introduction of salt solution from nothing to more than 2½ times the mean concentration during the test, as shown in Table 13. The conclusion is the same as that of Section 50, namely, that large variations in the concentration of salt in the tail-race make relatively small variations in the average concentration computed therefrom, even though there are considerable variations of velocity in the tail-race, as in the regular turbine tests where the velocity variations were from 1 or 2 ft. per sec. on one side to 4 or 5 ft. on the other.

The ratios above computed are somewhat improved, especially for all tests after the first, which latter took place just after opening the salt valve, by weighting the concentrations according to the scheme of weights given in the diagram, Fig. 37.

Thus, the weights from that diagram for the four halves of the two draft-tubes for Test *M*, in which the gate was set at 14.5, are

Upper west..................0.302	Upper tube....0.50
Upper east..................0.198	
Lower west..................0.320	Lower tube....0.50
Lower east..................0.180	

Possibly the discharge of the upper draft-tube should have slightly more weight than the lower one, as it is supposed that the gate of the upper turbine was slightly more open than that of the lower, but the diagram is based on equal weights.

Weighting the averages for each of the four quarters of the tail-race, accordingly, Table 15 has been compiled.

TABLE 15.—TESTS *M*.—WEIGHTED MEAN TITRATIONS OF HALF-LITER TAIL-RACE SAMPLES, PER LITER PER SECOND, OF SALT SOLUTION DISCHARGED FROM EACH OF THE SIX SPRINKLER PIPES IN THE HEAD-RACE, AND RATIO OF THE WEIGHTED MEAN TO THE ARITHMETICAL MEAN.

(Compiled from Tables 12 and 13.)

(The units are cubic centimeters of silver nitrate per liter per second of salt solution for half-liter tail-water samples.)

Number of sprinkler open.	Salt-rate, in liters per second.	Weighted mean, per liter per second, of salt solution.	Arithmetical mean, per liter per second, of salt solution.	Ratio of weighted to arithmetical means.
1	1.53	6.28	6.08	1.03
2	1.63	5.84	5.84	1.00
3	1.62	5.80	5.99	0.97
4	1.61	5.47	5.62	0.97
5	1.71	4.97	4 93	1.01
6	1.78	5.05	5.05	1.00
Average of all six		5.48	5.51	
Average of last five		5.33	5.39	

The last column might be computed by weighting the ratios given in Table 13, instead of taking the quotients of the weighted and arithmetical means directly as in Table 15.

It is seen from Table 15 that 1.03, the ratio for the first of Tests *M*, is not in the direction of improving the weighted mean for this test, and that the last ratio should be greater than, instead of equal to, unity. Otherwise, all the other ratios of the last column tend to reduce the variations among the weighted means, decreasing the arithmetical means which are larger than the average and increasing those which are smaller. For example, the maximum variation of the weighted mean is 5.80 — 4.97 = 0.83, and that of the arithmetical mean is 5.99 — 4.93 = 1.06. Had the tests been of longer duration,

this result would have been materially improved, and the variations among the values of the weighted mean would have tended to vanish altogether with increasing care in the experiments. Of course, we depend here, also, on the current-meter tests, on which Fig. 37 is based. There are no doubt errors of some magnitude in the current-meter work, but the error introduced into the weights has been shown not to be serious for the purpose of computing the discharge.

TABLE 16.—Observed and Weighted Concentrations of Salt in Tail-Race Due to the Separate Discharge from Each Quarter of the Head-Race Sprinkling System, Tests 101-104, Unit No. 11.

(The unit is the titration of 100 c.c. unevaporated tail-water. $AgNO_3$ = 1.56 grammes per liter.)

Test No.	Half of tail-race.	Actual titration of 100-c.c. sample of tail-water due to salt from tank only.		Weighted titration.	
		W.	E.	W.	E.
101 Upper west quarter of sprinkling system open...........	Upper... Lower...	8.99 8.68	12.35 11.19	3.49 3.27	1.39 1.38
	Means...	10.30		9.53	
102 Lower west quarter open.....	Upper... Lower...	3.28 13.27	4.52 15.63	1.27 5.00	0.51 1.92
	Means...	9.18		8.70	
103 Lower east quarter open.....	Upper... Lower...	2.71 13.32	3.19 12.25	1.05 5.02	0.36 1.51
	Means...	7.87		7.94	
104 Upper east quarter open......	Upper... Lower...	11.79 2.53	10.15 2.68	4.57 0.95	1.15 0.33
	Means...	6.79		7.00	

Several tests similar to those just discussed were made on Unit No. 11. These are Tests 101-104. In each case the sprinkler pipes of only one-quarter of the head-race were opened, the pipes in the other three-quarters remaining closed. The effect on the distribution is shown in Table 16.

In this case only 100 c.c. of tail-water constituted a sample, and this was titrated directly without evaporation. Thus, the titrations of these tests and those of Tests *M* are not directly comparable, the titrations of 100 c.c., unevaporated, being relatively considerably larger than those for 500 c.c. evaporated. Moreover, the titration of 100 c.c. of normal head-water directly requires from 3.50 to 4.50 c.c., which is relatively much larger than for 100 c.c. of tail-water, the comparable value probably being 3.50, which is adopted for the following discussion. This value has also been taken high on account of the fact that a 100-c.c. sample of tail-water will titrate several per cent. higher, relatively, than the half-liter samples of Tests *M*. By thus having the normal head-water titration too high, the remainder, after subtraction, will be decreased several per cent., which will tend to counteract the over-titration.

The weights for the four quarters of the tail-race for Tests 101-104 on Unit No. 11, from Fig. 39, are:

	West.	East.
Upper	0.387	0.113
Lower	0.377	0.123

When these weights are applied to the actual titrations of Table 16, the corresponding weighted titrations result.

By dividing these titrations by the salt rate and multiplying by 5 to reduce the titration to ½ liter, we should be able to make some comparison between the distribution tests on Unit No. 13, summarized in Table 15, and those on Unit No. 11. Thus Table 17 has been compiled.

These rates of titration are considerably larger than those of Tests *M*. This is due to the fact that the discharge was considerably larger in Tests *M* than in Tests 101-104. The titrations should also be proportional to the strength of the salt solution. The salt solution was not titrated, but 100 c.c. of the tail-water was titrated to about 11.8 c.c., unreduced by the initial salt content, which corresponds to

59 c.c. per ½ liter. The hydrometer readings for the salt solution in Test 105 was 1 180, and for Tests 101-104 it was 1 178, so that the salt solution was of the same strength as in Test 105 when the titration was at an average of 53.5. The titration of the salt solution in Tests *M* was 54.8, so that, admitting errors of 2 or 3%, the two tests will be comparable when the difference of discharge is taken into account. The discharges in the two cases are about as follows:

Tests *M*	1 730 cu. ft. per sec.	= 49 000 liters per sec.
Tests 101—104	1 510 " " " "	= 42 700 " " "

TABLE 17.—Tests 101-104.—Weighted Mean Titrations per Liter Per Second of Salt Solution Discharged from Each Quarter of the Sprinkler System in Head-Race of Unit No. 11 for 100-c.c. Samples of Tail-Water, and Ratio of the Weighted Mean to the Arithmetical Mean.

Quarter of head race treated with salt solution.	Discharge of salt solution, in liters per second.	Weighted mean titration. Table 16.	Arithmetical mean. Table 16.	Ratio of weighted to arithmetical mean.
Upper west (101).....	7.00	6.80	7.36	0.92
Lower west (102).....	6.85	6.35	6.70	0.95
Lower east (103).....	6.80	5.84	5.79	1.01
Upper east (104).....	6.80	5.14	5.00	1.03
Mean for first three tests...............		6.33	6.62	
Means for all tests........................		6.03	6.21	

The reader's attention has been called to the fact that the first and last of Tests *M* were questionable, and it is now also stated that the salt solution was exhausted in the middle of Test 104, so that this test was cut short, with the result that the incident casts doubt on the weighted mean for this test in Table 17. Cutting out these questionable tests, the average variations of the weighted and arithmetical means in Table 17 are, respectively, $96 \div 6.33 \backsimeq 15\%$ and $1.57 \div 6.62 \backsimeq 24$ per cent. Thus the variation of weighted values from their mean is only 7½%, although the variation of average titrations in the four quarters of the tail-race is, in some cases, 4, or 5, to 1.

To make a comparison of the mean titrations per liter per second of discharge from the salt solution tank in Tests *M* with those of Tests 101–104, therefore, we may with some degree of approximation take the product of the titrations by the discharges of the turbines

in the two cases. If these products are doubled they will represent the theoretical titration of the entire discharge of the turbine for 1 sec. when the salt rate is 1 liter of solution per sec., the density of this solution being about 1.18. The results are:

Middle 4 of Tests M $2 \times 49\,000 \times 5.39 = 528\,000$ c.c.;
First 3 of Tests 101–104 $2 \times 42\,700 \times 6.33 = 540\,000$ c.c.;

which agree reasonably well for the character of work in the distribution tests. The agreement would have been even better had the middle pair, which are the more trustworthy, of Tests 101–104, been taken instead of the first three. Moreover, these figures are just about what 1 liter of salt solution would give when titrated in the usual manner. For example, the titrations of 10 c.c. of a 100 to 1 dilution of salt solution with distilled water in the two tests have been stated to be

Tests $M = 54.8$, or, 548 000 per liter;
Tests $101\text{–}104 = 53.5$, or, 535 000 per liter.

Therefore, the relative variations of the weighted values from their means in Tests M and 101–104 are, respectively,

for Tests M $\frac{1}{2}$ of $0.83 \div 5.39 = 8\%$;
for Tests 101–104 $\frac{1}{2}$ of $0.96 \div 6.33 = 7\frac{1}{2}\%$;

which may be attributed in some measure to the variations of the concentrations at different parts of the tail-race, as shown by Tables 13 and 16, in which they are, respectively,

for Test M, from 2 to 1 to 100 to 1, in Unit No. 13;
for Tests 101–104, from $1\frac{1}{2}$ to 1 to 5 to 1, in Unit No. 11.

53.—*Precision Inferred from Distribution Tests.*—An estimate of the order of precision of the regular tests, as to constant errors due to imperfect mixing, may be made from the foregoing discussion. That is to say, we may infer approximately the degree of error in the average concentrations, determined on very carefully conducted tests, from the errors observed in the concentrations of very hasty rough tests.

Without taking extreme values, the following estimate of the variations of concentration of salt in the tail-water in the different quar-

ters of the tail-race, as shown by Tables 13 and 16 for Tests M and 101-104, respectively, are:

Tests M:	1....35 to 1	Test: 101....2 to 1
	2....10 to 1	102....5 to 1
	3.... 2 to 1	103....5 to 1
	4.... 4 to 1	104....5 to 1
	5....15 to 1	
	6....84 to 1	
Mean.........25 to 1		Mean........4 to 1

Now, if it is assumed that the precision varies directly with the foregoing mean variation, it will follow that a variation in concentration of n to 1 should, as an average, result in a precision giving rise to a systematic error of

$$\frac{8\,(n-1)}{24}\% \text{ for Tests } M, \text{ Unit No. 13} \qquad (139)$$

and an error of

$$\frac{7\frac{1}{2}\,(n-1)}{3}\% \text{ for Tests 101–104, Unit No. 11} \qquad (140)$$

We have no distribution tests for Unit No. 12, but it may be said that conditions on this unit were intermediate to those on Units Nos. 11 and 13, even to the fact that Unit No. 12 is situated between the other two. Variations of concentration in certain tests on Units Nos. 11 and 13 are shown in Table 18.

Thus, by Equations (139) and (140), we have as the error due to erroneous weighting of concentrations in the best tests on Unit No. 13, by this method,

$$\frac{8 \times 0.045}{24} = 0.015\%,$$

and, for the tests on Unit No. 11, the error is

$$\frac{7\frac{1}{2} \times 0.165}{3} = 0.41 \text{ per cent.}$$

These results are somewhat better than merely relative. They show that the error due to imperfect mixture is very small in the case of Unit No. 13, being negligible, in fact, and that we may expect errors on Unit No. 11 of an appreciable fraction of 1 per cent. The error in the final location of a plotted curve, however, should be less still than these approximate values, which are also to be considered too high, as they undoubtedly contain accidental components.

TABLE 18.—Approximate Maximum Variations of Concentration of Salt in the Four Quarters of the Tail-Race in Certain Tests on Units Nos. 11, 12, and 13.

Unit No. 11.		Unit No. 13.		Unit No. 12.	
Test No.	Variation.	Test No.	Variation.	Test No.	Maximum variation.
105	1.34	22	1.02	200	1.23
106	1.06	23	1.06	201	1.06
107	1.20	26	1.08	202	1.13
108	1.06	32	1.02	203	1.27
109	1.06	33	1.04	204	1.26
111	1.18	39	1.14	205	1.48
112	1.26	40	1.02	206	1.19
113	1.26	41	1.01	207	1.15
114	1.24	42	1.01	208	1.25
115	1.20	43	1.07	209	1.23
116	1.15	44	1.03	210	1.06
117	1.15	45	1.03	211	1.05
118	1.06	46	1.05	212	1.07
119	1.12				
120	1.05				
121	1.23				
122	1.19				
Means.....	1.165		1.045		1.187

54.—Time Required to Establish Steady Flow of Salt.—An idea of the variation of concentration for 2-min. averages may be gained from the results of taking a sample every 2 min. from one of the tail-race sampling pumps in Test *A*, July 11th, 1915. The first sample was taken just in time to avoid capturing any salt, immediately before opening the salt-solution valve in the head-race. The samples titrated as follows, in order of number:

1	1305	5	5160	9	5060	13	4930
2	5305	6	5005	10	4920	14	5430
3	5065	7	4995	11	4700	15	5105
4	5090	8	4990	12	4700	16	5100

Units are in hundredths of cubic centimeters titration per ½-liter sample evaporated.

The variations of the concentration from interval to interval are due partly to variations of the rate of introducing the salt solution and partly to those irregularities of the discharge of the turbine which may be expected for intervals as short as 2 min. That is, there are surges in the head-race which produce irregularities of concentration, even

when the salt rate is constant. There was not a sufficiently close connection between time and concentration in Test *A* to admit of a more complete discussion. The important fact to be noticed in this test is that it required but one time interval, 2 min., for the concentration to jump from that of normal head-water, 13.05 c.c., to fully concentrated tail-water, 53.05 c.c. The reason that this latter titration is considerably larger, with only one exception, than all the others is that the salt-solution valve is first fully opened and then adjusted to some predetermined salt rate.

In Test *D* there is a better connection between the time of starting the salt and the time of steady concentration. With possible errors of ½ min., the results are as given in Table 19.

TABLE 19.—Two-Minute, Half-Liter, Tail-Water Samples in Test *D*.

Time, P. M. Hour and minute. H. M.	Salt rate during interval preceding, in liters per second.	Titration of tail-water sample, in hundredths of cubic centimeters.	Titration, less 1270, for normal head-water.	Remarks.
2-54	....	.....	.....	Began taking 2-min samples.
56	0.00	1 270	0	
58	0.00	1 275	0	Started salt solution.
3-00	6.41	1 350	80	
2	6.68	4 200	2 930	
4	6.90	4 815	3 545	
6	7.08	4 620	3 350	
8	7.28	4 725	3 455	
10	7.48	5 080	3 810	
12	7.68	5 150	3 880	
14	7.79	5 195	3 925	
16	7.93	5 320	4 050	
18	7.72	5 460	4 190	
20	7.68	5 530	4 260	
22	7.68	5 660	4 390	
24	....	5 590	4 320	

The tests in Table 19 indicate fairly well that it required about 5 or 6 min. for the salt solution to adjust itself to steady conditions after opening the valve. The same conclusion may be drawn from the time required for the decrease in salt rate during the interval from 3-16 to 3-18 to make itself felt in a corresponding decrease of tail-water titration for the interval 3-22 to 3-24. Again, if all the titrations in the fourth column of Table 19 are divided by the salt rate for the interval preceding it by 6 min., a fairly constant ratio results, showing that 6 min. seems to represent the time required for conditions of salt flow

to adjust themselves to changes of a more or less sudden nature. Beginning with the titration, 3 350, for 3-06 P. M., dividing it by the salt rate, 6.41, for 3-00 P. M., two figures of the ratio may be tabulated as follows:

52, 52, 55, 55, 54, 54, 54, 55, 55, 56.

TABLE 20.—MINUTE TAIL-WATER SAMPLES, TEST *U*.

Time. H. M.	Salt rate during preceding interval, in liters per second.	Titration of tail-water sample, in hundredths of cubic centimeters.	Titration, less 1330, for normal head-water, in hundredths of cubic centimeters.	Remarks.
4-35	....	.....	.....	Started minute samples.
36	....	1 345	0	
37	....	1 350	0	
38	....	1 310	0	Started salt solution.
39	?	1 315	0	
40	?	1 430	100	Half-liter samples taken from Pump 25.
41	8	5 300	3 970	
42	8	6 695	5 365	
43	8	6 510	5 180	
44	8	6 485	5 155	

In Test *U*, again, it is seen that it requires 5 or 6 min. for the conditions of flow to become steady. The samples were not continued long enough to make the resulting rate of titration determinable, and the salt rate is uncertain during the first 2 min. of discharge. The final ratio of titration to constant salt rate, carried out to two places, is about 65 (5 160 ÷ 8). This figure could be compared with the ratio, 55, in Test *D*, by taking account of the strength of the salt solution and discharge in the two tests involved. The relative location of points at which samples were taken and the distribution of salt as injected in the head-race are also of moment.

The samples in Table 21 were taken at the end of tests to permit of observations on the weakening concentration when the flow of salt solution is stopped.

In Test 22 it appears that it requires perhaps 8 min. for the salt to disappear completely from the tail-race. The exact instant of closing the salt valve, however, is not recorded, so that there may be an error of a minute or so in the time.

It may be concluded from these results that it requires about 6 or 7 min. to establish uniform conditions of salt flow after changing the

TABLE 21.—TEST 22.—TWO-MINUTE TAIL-WATER SAMPLES.

Time. H. M.	Salt rate during preceding interval, in liters per second.	Titration of tail-water sample on ½ liter.		Titration, less 1310.		Remarks.
		W.	E.	W.	E.	
3–58	7.1	......	......	......	......	W. samples came from Pump 41 C. L. upper draft-tube.
4–00	?	4 005	4 875	2 695	3 565	E. samples came from Pump 42 C. L. of lower draft-tube.
2	0	1 680	1 755	370	445	Stopped salt some time after 3–58.
4	0	1 325	1 410	15	100	Started samples 3–58.
6	0	1 325	1 325	15	15	Salt stopped some time between 3–58–30 and 3–59–30.
8	0	1 340	1 310	30	00	
10	0	1 310	1 305	00	00	

valve or otherwise altering circumstances in the head-race which produce changes of concentration in the tail-race.

55.—Difficulty in Securing Tail-Race Samples.—It was at first feared that in the tail-race there might be eddies and reverse currents of such a nature as to cause the samples taken from the sampling pumps to be contaminated. It was also feared that there might be reverse currents in the head-race which would allow salt to escape into the head-water of the adjacent units. In either case, the tail-water samples would not be representative.

The first of these sources of error was investigated by shoveling a ton or more of salt into the tail-water at the mouth of the tail-race, some 7 or 8 ft. down stream from the sampling pipes, during which time the sampling pumps were kept in operation. No salt whatever was found in the tail-water samples, thus proving that the water was all flowing outward and that no contamination of samples took place. The actual titrations of ½-liter samples are given in Table 22.

The writer has had experience in tests, however, where the draft-tubes and tail-race conditions were not as good as in the case described, the result being that samples of tail-water became instantly contaminated when salt was thrown into the tail-race at a considerable distance down stream from the sampling pipes. In such cases the sampling pipes should be moved back as close to the runner as possible. Even then there will no doubt be occasional cases where it will be found necessary to take samples in the head-race before the feed-water passes through the runner. In this event it will be necessary to take extraordinary precautions to secure a uniform mixture at the distributing system in the head-race, or as quickly after the salt solution is injected into the head-water as possible.

TABLE 22.—TEST 47.—TITRATIONS OF TAIL-WATER BEFORE AND DURING THE INTRODUCTION OF SALT INTO TAIL-WATER 7 OR 8 FT. DOWN STREAM FROM SAMPLING PIPES IN TAIL-RACE.

(Units are $\frac{1}{100}$ c.c. titration of $\frac{1}{2}$ liter of tail-water.)

BEFORE SHOVELING SALT INTO TAIL-WATER.

1330		1314		1320	
	1314		1305		1330
1820		1303		1305	
	1300		1305		1305
1290		1295		1320	
	1295		1320		1312

DURING THE SHOVELING OF SALT INTO TAIL-WATER.

1305		1300		1323	
	1295		1302		1310
1297		1298		1298	
	1295		1300		1310
1321		1297		1295	
	1300		1290		1293

MEANS FOR THE FOUR QUARTERS OF TAIL-RACE FROM ABOVE FIGURES.

	Before introducing salt.		During introduction of salt.	
	West.	East.	West.	East.
Upper	1 316	1 315	1 299	1 308
Lower	1 295	1 312	1 303	1 297

Racks Expedite Mixing.—When uniform mixtures must be effected in the head-race as quickly as possible, it will be found that racks placed in the head-race will assist materially. In many cases the distributing pipes may be placed immediately down stream from the usual trash racks, thus making economical use of the power-house installation. However, the openings of the trash racks are entirely too large to raise sufficient head, and it will be necessary, when the trash racks are to be used in this way, to reduce the openings by suitable fillers, or by horizontal strips across the racks uniformly spaced in the vertical direction.

The head which it is necessary to raise to render the velocity uniform in all parts of the section down stream from the racks will depend on local conditions. Indeed, it may be desirable to have this head adjustable, as in the case of a contract specifying the head on the turbines during the tests. In such a case the openings between the strips must be adjustable.

By changing the head lost through the racks, the head on the turbines can be regulated.

The head raised in such cases must be sufficient to render the velocity of the water practically uniform over the entire cross-section of the head-race. This will require a head of not less than 1 ft. on the racks and even more when the velocities in the head-race in its normal condition are very irregular over the cross-section. Racks raising heads of 4 or 5 ft. are not difficult to construct.

Designers of stilling racks have frequently failed to make the openings in the racks small enough. If 1-in. planks are used, it will generally be found that they should be less than 1 in. from center to center. Overloading must be avoided when raising heads on trash racks.

56.—Efficient Mixing, and Design of Sprinklers.—In order to secure this desirable result, it will be necessary to have as many perforations in the distributing sprinklers as economy and operating conditions will permit. Practically, the size of the holes will be limited by the quantity and quality of the water taken up by the centrifugal pump, and the character of the scale which breaks from the distributing pipes themselves. If the pipes are fitted with flushing outlets, say T's, equipped with valves at the middle or end of each sprinkler, any accumulation of particles of scale or other materials which tend to plug the holes might be occasionally flushed out, but even then some of the holes may become clogged and require individual attention. The safe rule to follow is to have a large centrifugal pump, so that the holes may be made both numerous and large, thus not only preventing the clogging of the holes but also improving the mixture by allowing the centrifugal pump to do a great share of the mixing.

E. W. Schoder, Assoc. M. Am. Soc. C. E., of Cornell University, has made some experiments which will assist in designing efficient sprinklers. The results are given in Table 23.

The diameters given in Table 23 refer to the size of the drill. The diameters of the holes were not measured. The holes were drilled in 2-in. black pipe, and no attempt was made to remove burrs on the inside of the pipe.

The discharges given in Table 23 were measured with jets singly in action, with the tank water surface about 22 ft. above the orifices. A measure of the loss of head in the pipe leading from the tank to the orifices is given by the measured discharge with the orifice pipe

TABLE 23.—CHARACTER OF DISCHARGE FROM HOLES IN 2-INCH PIPE SPACED 2 INCHES APART. FROM EXPERIMENTS BY PROFESSOR E. W. SCHODER.

Normal size of holes.	Diameter of holes, in inches.	Discharge, in liters per minute.	Head, in feet.
5/16 in.	0.312	19.4	19.6
7/32 in.	0.219	10.3	21.5
No. 10	0.194	8.28	22.0
No. 20	0.161	7.46	22.5
No. 30	0.128	4.58	22.5
No. 40	0.098	2.72	22.5
No. 50	0.070	1.31	22.5
No. 60	0.040	0.222	22.5
3/8 in	0.375	26.6	17.4

removed. With a difference of level of 20.5 ft., the discharge was 55.5 liters per min., the velocity in the 1-in. pipe being about 6 ft. per sec. We may take 20 ft. as the loss of head for this discharge. With the loss taken proportional to the square of the discharge, the $\frac{3}{8}$-in. orifice discharge caused 4.6 ft. loss of head, leaving 17.4 ft. as the approximate effective head. The heads given in Table 23 are based on such calculations.

A photograph of all the jets except the $\frac{3}{8}$-in., discharging simultaneously vertically upward, shows the relative heights reached in air, as well as the orifice pipe arrangement. Another photograph shows the tank, the pipe line, and the orifice pipe resting on the bridge spanning the canal.

For the tests on the jets submerged in flowing water, $\frac{1}{2}$ lb. of Prussian blue was dissolved in about 500 gal. of water. This gave an intensity such that one's finger nail became invisible when submerged $1\frac{1}{4}$ in. beneath the surface in bright sunlight.

During the tests with the colored water, the velocity of flow in the canal was 1.8 ft. per sec. where the jets issued.

The jets were tested issuing horizontally at right angles to the direction of flow in the canal and also issuing vertically. The pulsations in the canal flow caused a lateral swaying of the colored stream at from about 2 to 4-sec. intervals.

When issuing vertically upward, the 2-in. orifice pipe being perpendicular to the flow in the canal, a sample of water near the surface about 4 ft. down stream from the orifice was caught in a bucket. By noting the intensity of the color of this sample some indication

of the diffusion was obtained. Owing to the large relative dilution, the method fails for small orifices.

In all cases the color seemed to be uniformly distributed across the band for several feet down stream. Below about 6 ft. down stream, the flow pulsations caused localization in the colored stream in swellings of the colored band and separation into two bands. The general appearance, however, was that of a wavy, slowly widening band of blue, somewhat like smoke from a chimney. Table 24 gives the measurements.

TABLE 24.

Orifice.	Direction.	Submergence, in inches.	Distance down stream from orifice, in inches.	Distance out from plane of orifice of edges of strongly colored stream, in inches.	Instantaneous width of colored stream, in inches.	Color of sample 4 ft. down stream usually from the surface to 6 in. below.
3/8-in.....	Horizontal.	11½	2	3 to 9		
			6	5½ to 11½		
			30	4 to 17		
	Vertical...	16	24		6 to 12	Strong at surface.
		15	9	(Boils up to surface.)		
5/16-in...	Horizontal.	11½	2	4 to 8		
			12	6 to 12		
			42	9 to 18		
	Vertical...	16	26		7½	About same color top 6 in. and top 12 in.
			8 ft.	15 to 18		
7/32-in...	Vertical...	16½	26	6		None.
			8 ft.	12 to 15		
		10½				Good.
	Horizontal.	11½	2	4 to 8		
			12	8 to 13		
			42	11 to 21 and 9 to 20	6 to 7	
No. 10...	Horizontal.	6	2	3½ to 7		
			12	7 to 12		
			40	9 to 18	6	
	Vertical....	16	26		5 in. through 8-in. range, none sidewise.	
		10½				Faint.
No. 20...	Horizontal.	6	2	4 to 7½		
			12	5½ to 12½	4	
			38	9 to 17½	5	
No. 30...	Horizontal.	6	2	3 to 6		
			12	5½ to 9	3	
			36	5 to 13	4 to 5	
No. 40...	Horizontal.	6	2	3 to 5		
			8	3½ to 8	2½	
			36	5 to 11	3	
No. 50...	Horizontal.	6	2	2½ to 4½		
			6	2½ to 6		
			32	Middle of faint band about 5 in. out.		
No. 60...	Horizontal.	6	2	1½ to 2		
			4	1½ to 4½	1	

57.—As an illustration, it may be observed that the velocity of the water in the canal as it flowed past the orifices in the pipe was about 1.8 ft. per sec., or approximately what it would be in an ordinary open

head-race. The object is to determine the sizes, spacing of perforations, and spacing of sprinkling pipes, so as to obtain as uniform a mixture as possible at the time of injection, thus assisting the mixing as much as possible at the outset.

Fig. 2 shows an efficient system of holes, based on the results of the experiments just described, but, undoubtedly, this can be improved when more extensive experiments are made.

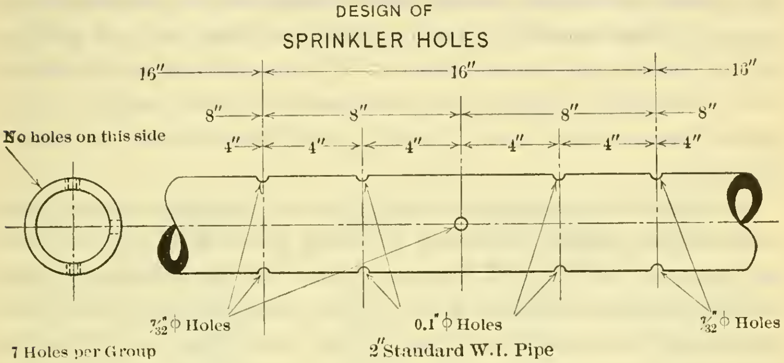


FIG. 2.

The following are the data for a design for sprinkler holes: Intake velocities, 1.8 ft. per sec.; head on sprinkler orifices, 22.5 ft.; discharge of $\frac{7}{32}$-in. orifice, 10.5 liters per min.; discharge of No. 20 drill-hole, 7.46 liters per min.; discharge of No. 40 drill-hole, 2.72 liters per min.; distances out to edges of band supplied with salt by $\frac{7}{32}$-in. hole, 11 and 21 in.; the same for No. 20 hole, 5 and 13 in.; the same for No. 40 hole, 5 and 11 in.

TABLE 25.—DIMENSIONS.

Number of holes.	Diameter of drill, in inches.	Discharge for 22½-ft. head, in liters per second.	Distance to edges of salt band, in inches.	Width of band served, in inches.	Supply of solution per inch of width, in liters per minute.	Direction of discharge of hole.	Position of hole in pipe.
1	7/32	10.5	+11 to +21	10	1.05	Up.	Top.
2	0.098	5.44	+ 5 to +11	6	0.91	Up.	Top.
1	7/32	10.5	− 5 to + 5	10	1.05	Up stream.	Up-stream face.
2	0.098	5.44	− 5 to −11	6	0.91	Down.	Bottom.
1	7/32	10.5	−11 to −21	10	1.05	Down.	Bottom.
Total		44.78					

It will be seen from Table 25 that it is intended to keep the holes under a head of 22.5 ft., and that each sprinkler extending across the head-race at right angles to the current is to supply a band of salt 42 in. wide, extending 21 in. above and below the center-line of pipe. (This neglects the diameter of pipe.) Hence the spacing between the sprinklers should be 42 in. This spacing can be altered by changing the pressure on the orifices corresponding to the desired alteration. The spacing of the holes depends on the discharge from the centrifugal pump. If the centrifugal pump discharges 1 000 gal. per min. at 22.5 ft. head on the orifices (that is 3 780 liters per min.), there would be required 3 780 ÷ 44.78 = 84, or 85, groups of holes which, with the size of head-race and depth of water, would determine the spacing of the groups.

It is seen that this does not effect a perfect distribution upward and downward, the supply of solution per group per inch of width of band varying from 0.91 to 1.05 liters per min., but this irregularity may be more apparent than real, owing to the uncertainties connected with reproducing the exact conditions of the experiments on which the design is based. The main object will be secured if the spacing of groups is close enough, and if the orifices distribute the salt solution well over the entire cross-section of the head-race. There is room for more experimentation on this subject, for, in cases where the tail-water samples must be taken in the head-race, thus necessitating the best possible mixture immediately after the injection of the salt solution, it is imperative to have nice work in the design and operation of the distributing system.

The actual design of the sprinkler orifices is shown by Fig. 3. As there was no special difficulty in securing mixtures in these tests, little attention was paid to rigid rules for the design and operation of the sprinkler system. The orifices were usually under a head of about 40 ft., so that it is hard to state from the results of the orifice tests just how thoroughly the sprinkler system effected a mixture independently of the turbines. A better idea on this point may be secured by examining the results of the distribution tests which have been discussed in Section 52, *et seq.*

58.—Discharge of the Orifices in Tests.—It was intended to follow the general method just discussed for the design of the sprinklers in the tests, but it is believed that the holes were not exactly of the sizes

originally contemplated. Moreover, more or less loose scale in the pipes tended to stop up the holes, thus interfering at times with a full realization of the discharges anticipated. The approximate results for one group of holes are shown in Table 26, from which it will be seen that the total discharge of the eight orifices, under 22.5 ft. head, should be 31.24 liters per min., or about $31.24 \times \sqrt{40 \div 22.5} = 41.6$ liters per min. per group of holes under 40 ft. head. There were 120 groups with one hole of extra small size at each end of each pipe, totaling, say, 121 groups, under an effective head of 40 ft., having a calculated capacity of

$$41.6 \times 121 = 5\,050 \text{ liters (or } 1\,335 \text{ gal.) per min.}$$

TABLE 26.

No. of holes.	Direction of discharge.	Diameter, in inches.	Discharge for 22.5 ft. head, in liters per minute.	Band of salt solution, in inches out.
1	Up.	0.161	7.46	+9 to +17½
2	Up.	0.098	5.44	+5 to +11
2	Horizontal.	0.098	5.44	−5 to + 5
2	Down.	0.098	5.44	−5 to −11
1	Down.	0.161	7.46	−9 to −17½
Total			31.24	

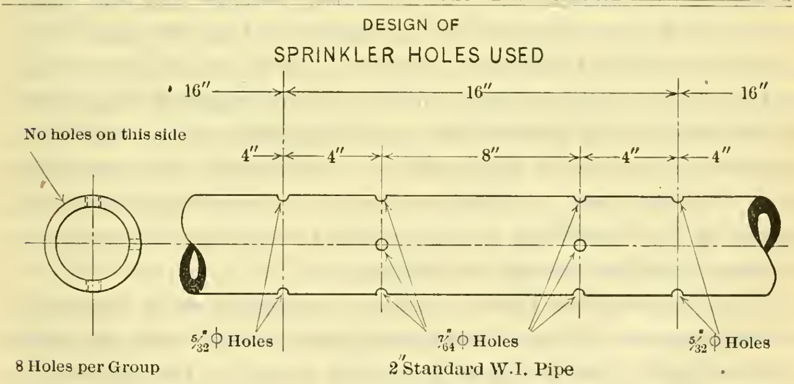


FIG. 3.

The holes, apparently, were of a somewhat different diameter than stated above, but the fact that the discharge of the centrifugal pump, as given in Section 51, and therefore of the sprinkler system, was always less than 1 000 gal. per min., would indicate that a considerable portion of the area of the holes was plugged in the majority of cases

59.—Leaking Pipes.—Another difficulty experienced in getting representative samples of tail-water was caused by leaks in the ¼-in. risers leading up from the tail-water. The crown of the tail-race arch was from 6 to 8 ft. under water, causing a vertical eddy flowing toward the power-house and against the face of the tail-race wall. This eddy necessarily carried a large portion of tail-water from the adjacent units and river, thus diluting the water next to the power-house wall for a depth of at least 6 ft. As the ¼-in. tail-sample pipes passed up through this water in the early tests, several of the pipes which became leaky on one or two occasions caused a contamination of the samples. which necessitated rejecting the corresponding titrations at such times.

After Test 23, however, the tail-race sampling pipes were moved back into the draft-tube, so that all remaining tests were free from this trouble, the pipes being entirely immersed in the tail-water from the turbine being tested.

60.—Error 7.—Errors Due to Variations of Temperature.—If the temperature varies during a test or series of tests, it will cause changes in the capacities of the measuring instruments, in the properties of the solutions, and in the determinations of the end points of titrations. Hence, it is necessary, when volumetric measurement of samples is used, to observe the temperatures of all the solutions, and of the air and water, both in the laboratory and on the actual test. (See Section 78 and Table 41.) It is then easy to compute the volumes and properties of the various solutions at some standard temperature, which standard had better be the mean temperature of the water flowing through the turbine during the test. This makes the computation of the discharge easy, as it does not require a correction to the volume computed, as this volume was computed at the actual temperature of the water flowing through the turbine.

It will be seen that there is a great advantage in using weights for the measurement of the salt solution injected into the head-race, rather than volumes. This is due to the simple nature of the equations in which weight is used, thus, in Equations (14) and (15), the titrations, p, p_1, p_2, and the rate of weight, W, of salt solution discharged into the head-race, are all that is necessary to know in order to determine W_2, the weight of the water discharged by the turbine. Again, it is this weight which is required to determine the theoretical power with which the turbine is to be charged in computing the efficiency. These equations

are nearly independent of temperature as the percentages, p, p_1, and p_2, determined by titration, are the same at all temperatures.

For these reasons, the method of weights involved in Equations (5) to (28), inclusive, is recommended as being better suited to turbine testing than the method of volumes used in the later equations. This does not mean that the silver nitrate is weighed. Indeed, the chemical determination is still volumetric, in that the titrations are conducted in exactly the same manner as before, the only difference being that the sizes of samples are determined by weighings.

Finally, any changes in the end-point, or strength, of the silver nitrate, due to changes in temperature during treatment in the laboratory, will be eliminated by the method of special dilutions and group titrations explained in connection with Errors 1 and 2, Sections 46, 47, and 48.

61.—Error 8.—Instrumental Errors.—It is scarcely necessary to enter at length into a discussion of the errors and their net effect which constitute Error 8. There are two errors of this nature, however, to which great attention must be given by the engineer in charge of the test, and these two only will be discussed somewhat in detail. These are the error likely to occur by reason of a change in the capacity, and consequently of the calibration, of the salt-solution tank; and the error due to changes of discharge of the pipettes used in the laboratory. It appears that the latter is not fully discussed in chemical literature, and it will therefore receive attention here. Of course, the engineer in charge of the test will assure himself that all instruments are properly graduated, calibrated, and placed so as to secure the desired degree of accuracy.

Pipette Errors.—Soon after the beginning of the turbine tests it was discovered that the pipettes used in the work changed their discharges with variations in temperature considerably more than had been expected. Rough field observations were at once instituted to discover the causes and to supply a method for determining as far as possible the effect of changes in temperature on their discharge. These tests showed at once that the discharge changed by several times the change in volume of a 10-c.c. pipette, due to a given change in temperature in the neighborhood of 20° cent. After due reflection it was concluded that the change in viscosity, surface tension, cohesion, etc., had a preponderating influence, when the temperature changed,

so as to augment the change in discharge far beyond what the mere change in volumetric capacity of the pipette would account for.

Moreover, it was found, as had been expected, that the concentration of salt in solution had a relatively large influence on the discharge of a pipette, and it became necessary to investigate this source of error, as well as the former. Accordingly, the pipettes were sent to the U. S. Bureau of Standards, Washington, after the turbine tests had been concluded, with the request that they be calibrated with distilled water and salt solutions of 150, 250, and 300 grammes concentration per liter. To ascertain the effect of temperature, each pipette was to be calibrated at three different temperatures, 15°, 20°, and 25°, cent., for its discharge of each of the four solutions specified. By these calibrations it was hoped to discover the law of variation.

Actual calibrations of the pipettes were made by the Bureau only at 20° cent., and the capacities at 15° and 25° cent. were then computed on the basis that the increment of discharge was equal to the increment of volumetric capacity, which was the very thing these desired calibrations sought to avoid. However, it was finally decided to make practical calibrations at the Carnegie School of Technology, and these were carried out with considerable success, by actually comparing the volume discharged by a pipette with a fixed volume.

The method of conducting one of these tests was, first, to find a place where a fairly constant temperature could be secured, and second, to discharge the pipette into a burette, the capacity of which had been previously ascertained by calibration. The solution used was made up to a known concentration of salt, except where it was required to test the discharge of distilled water. By making a number of such tests at various temperatures with each solution, it was possible to determine the effects of changes in temperature and concentration.

Fig. 4 shows the results of these tests in diagrammatic form, and Table 27 gives the actual observations. The use of the diagrams will be readily understood, as the discharge of any given pipette can be found at a point on the corresponding diagram opposite the known temperature and vertically above the concentration of the salt solution used, distilled water, of course, being indicated by zero concentration.

62.—Calibration of Salt Solution Tank.—In order to secure the desired degree of accuracy in measuring the salt solution consumed during a test, the solution tank must be so high that a vertical gauge-

board, of sufficient length, can be attached to the side, to admit of readings of the level of the solution in the tank with the necessary precision.

As it will ultimately be required to know the time rate at which the turbine develops power, it will also be necessary to note the time during which the surface of the salt solution falls from one level to another.

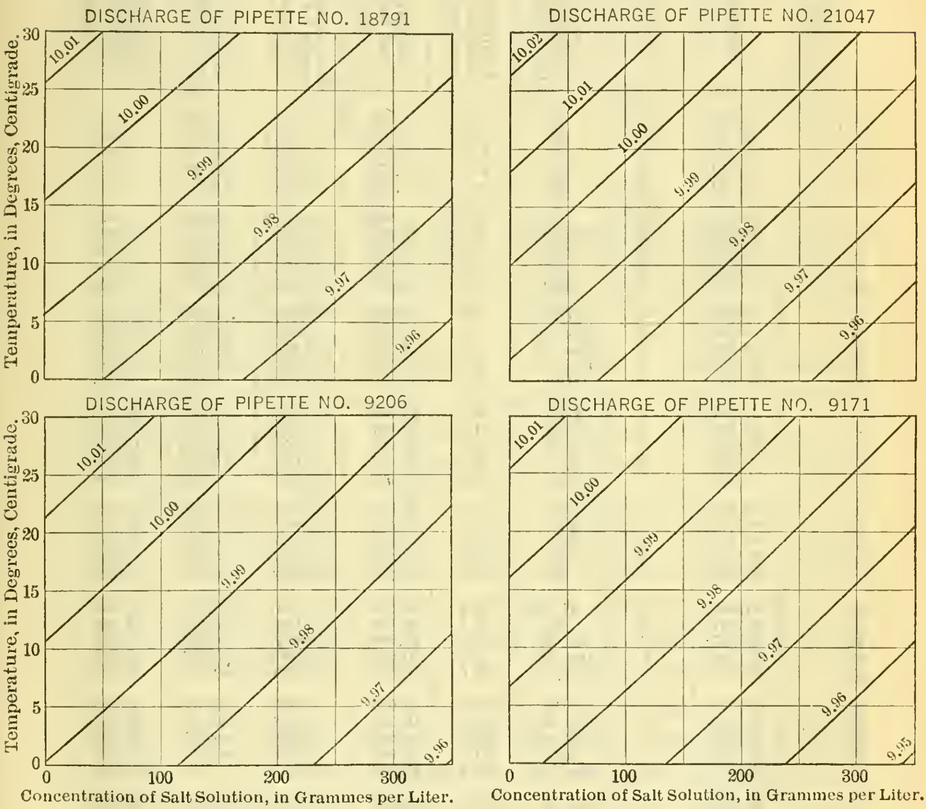


FIG. 4.

This time interval must be determined with the same precision as the quantity of salt solution consumed. Hence, it will be the duty of the engineer to determine the size of the tank and the length of the test, and these two elements will depend on the accuracy with which the timepiece and the tank-gauge can be read, and on the predetermined limit of permissible error.

TABLE 27.—Pipette Calibrations Corrected.

Taking Burette No. 6 = 100.070 c.c. at 20° cent.
" No. 4 = 100.090 c.c. at 20° cent.
" No. 3 = 100.05 c.c. at 20° cent.

Observer (Pipetted by).	Pipette No.	Burette No.	Strength of solution, in grammes per liter.	Temperature of solution, in degrees, centigrade.	Discharge as read, in cubic centimeters.	Temperature correction for burette volume, in cubic centimeters.	Correction on account of error, at 20° cent., in cubic centimeters.	True discharge at temperature of solution, in cubic centimeters.
Y........	18 791	6	300	24.6	9.973	+0.0012	+0.007	9.981
G........	18 791	6	300	25.2	9.973	+0.0013	+0.007	9.981
Y........	18 791	6	275	24.6	9.977	+0.0012	+0.007	9.985
G........	18 791	6	275	24.6	9.977	+0.0012	+0.007	9.985
Y........	18 791	6	250	24.4	9.980	+0.0011	+0.007	9.988
Y........	18 791	6	150	24.7	9.9865	+0.0012	+0.007	9.995
Y........	18 791	6	Dis. W.	21.1	10.000	+0.0010	+0.007	10.008
S........	18 791	6	300	1.2	9.960	—0.0047	+0.007	9.962
G........	18 791	6	300	1.0	9.958	—0.0050	+0.007	9.960
S........	18 791	6	250	0.75	9.966	—0.0050	+0.007	9.968
G........	18 791	6	Hyd.	2.0	9.982	—0.0045	+0.007	9.984
S........	18 791	6	Dis. W.	0.6	9.973	—0.0051	+0.007	9.975
S........	18 791	4	Dis. W.	1.2	9.982	—0.0047	+0.009	9.986
G........	18 791	4	Dis. W.	2 1	9.982	—0.0045	+0.009	9.986
S........	21 047	4	Hyd.	2.0	9.990	—0.0045	+0.009	9.994
S........	21 047	4	Hyd.	22.5	10.006	+0.0006	+0.009	10.016
S........	21 047	4	Hyd.	21.5	10.007	+0.0004	+0.009	10.016
S........	21 047	4	Hyd.	23.0	10.006	+0.0007	+0.009	10.016
S........	9 206	4	Hyd.	19.5	10.007	—0.0001	+0.009	10.016
S........	9 206	4	Hyd.	23.0	10.000	+0.0007	+0.009	10.010
S........	*9 171	4	Hyd.	24.0	10.012	+0.0010	+0.009	10.022
S........	*9 171	4	Hyd.	24.0	10.004	+0.0010	+0.009	10.014
S........	*9 171	4	Hyd.	23.0	10.000	+0.0007	+0.009	10.010
S........	21 047	4	250	23.0	9.976	+0.0007	+0.009	9.986
S........	21 047	4	300	23.0	9.964	+0.0007	+0.009	9.974
H........	21 047	3	N. H. W.	15.7	10.000	—0.0011	+0.005	10.004
H........	21 047	3	275	15.7	9.978	—0.0011	+0.005	9.982
H........	21 047	3	Dis. W.	15.8	10.008	—0.0011	+0.005	10.012
H........	21 047	3	Dis. W.	19.0	10.010	—0.0002	+0.005	10.015
H........	21 047	3	N. H. W.	20.0	10.004	0.0000	+0.005	10.009
H........	21 047	3	275	20.0	9.980	0.0000	+0.005	9.985
H........	21 047	3	Dis. W.	20.0	10.006	0.0000	+0.005	10 011
H........	21 047	3	N. H. W.	20.0	10.004	0.0000	+0.005	10.009
H........	21 047	3	275	20.0	9.980	0.0000	+0.005	9.985
H........	21 047	3	Dis. W.	23.8	10.020	+0.0009	+0.005	10.026
H........	21 047	3	Dis. W.	23.8	10.020	+0.0009	+0.005	10.026
H........	21 047	3	275	23.8	9.980	+0.0009	+0.005	9.986
H........	21 047	3	275	23.8	9.980	+0.0009	+0.005	9.986
H........	21 047	3	275	25.0	9.980	+0.0012	+0.005	9.986
H........	21 047	3	Dis. W.	25.0	10.022	+0.0012	+0.005	10.028
H........	21 047	3	N. H. W.	25.0	10.006	+0.0012	+0.005	10.012
S........	21 047	4	Hyd.	22.0	10.000	+0.0005	+0.009	10.010
S........	21 047	4	Hyd.	22.5	10.000	+0.0006	+0.009	10.010
S........	9 206	4	Hyd.	23.0	10.002	+0.0007	+0.009	10.012
S........	9 206	4	Hyd.	23.0	10.002	+0.0007	+0.009	10.012

Note.—Dis. W. = Distilled water. Hyd. = Hydrant water. N. H. W. = Normal head-water.

TABLE 27.—(*Continued.*)

Observer (Pipetted by).	Pipette No.	Burette No.	Strength of solution, in grammes per liter.	Temperature of solution, in degrees, centigrade.	Discharge as read, in cubic centimeters.	Temperature correction for burette volume, in cubic centimeters.	Correction on account of error at 20° cent., in cubic centimeters.	True discharge at temperature of solution, in cubic centimeters.
S........	18 791	4	Hyd.	23.0	9.998	+0.0007	+0.009	10.008
S.........	18 791	4	Hyd.	24.0	9.998	+0.0010	+0.009	10.008
S.........	9 171	4	Hyd.	23.0	9.990	+0.0007	+0.009	10.000
S.........	9 171	4	Hyd.	23.5	9.996	+0.0008	+0.009	10.006
S.........	21 047	4	Dis. W.	21.7	10.000	+0.0004	+0.009	10.009
S........	9 206	4	Dis. W.	21.5	10.004	+0.0004	+0.009	10.013
S.	18 791	4	Dis. W.	21.8	9.998	+0.0004	+0.009	10.007
S.........	9 171	4	Dis. W.	21.3	10.000	+0.0003	+0.009	10.009
S.........	9 171	4	Dis. W.	21.9	9.994	+0.0005	+0.009	10.004
S.........	21 047	4	300	21.7	9.974	+0.0004	+0.009	9.983
S.........	21 047	4	300	22.3	9.968	+0.0006	+0.009	9.978
S.........	9 206	4	300	22.5	9.976	+0.0006	+0.009	9.986
S.........	9 206	4	300	22.5	9.974	+0.0006	+0.009	9.984
S.........	18 791	4	300	22.6	9.968	+0.0006	+0.009	9.978
S.........	9 171	4	300	22.6	9.968	+0.0006	+0.009	9.978
S.........	9 171	4	300	22.5	9.966	+0.0006	+0.009	9.976

NOTE.—Dis. W. = Distilled water. Hyd. = Hydrant water. N. H. W. = Normal head-water.

The tank used in the tests described was about 10 ft. high, and a little less than 10 ft. in diameter. A gauge-board, carrying an ordinary gauge-scale, graduated to feet and tenths of feet, was attached to the side of the tank, and a 1-in. glass tube was set in a slot along the axis of the board. This tube was connected with the tank at the bottom by a piece of rubber hose, with a nipple and valve. A second valve, or pet-cock, was provided to admit of emptying the glass tube while the valve to the tank was kept closed. This was necessary in order to insure the uniformity of solution in the tank and tube. It may be necessary to flush out the tube more than once during a test, pouring the solution thus drawn off back into the tank. This would be the case where there was a great difference in temperature between the solution in the tank and the air surrounding the glass tube. Otherwise, the solution in the glass gauge would soon have a density different from that in the tank, thus indicating false surface levels.

On the face of the board, opposite the edge graduated in feet and tenths of feet, a calibration scale was marked off with a lead pencil and carpenter's square. This calibration was made by filling the tank with water and then drawing out equal volumes successively, making a straight horizontal calibration mark after each draft of the standard

volume. This standard volume may be constructed out of a piece of 12-in. pipe, or the volume may be determined from a weighing, correcting for temperature variations, in which case an investigation must be made to ascertain whether the water used has any serious deviation in its density from that of distilled water. This latter statement, however, is made only when considering the calibrator as representing a true unit of volume. It is not necessary, for the purpose of efficiency tests, to have the tank or calibrator calibrated to deliver true volumes by weighings of distilled water. The actual feed-water may be used in place of distilled water, any error made in assuming its density to be that of distilled water being compensated by the counter error committed when the theoretical power of the feed-water is calculated on the basis that it is distilled water. Thus, the product of the observed volume, too large in a certain ratio, and the density of distilled water, too small in a like ratio, occur in the formula for efficiency. Therefore, the efficiency is correctly determined, although the discharge derived from the tank observations may be slightly in error. This question is more fully discussed under Error 10, Section 73.

63.—When Does a Test Begin and End?—In operating this tank for a test, the regulating valve leading to the distributing pipes was opened about 7 or 8 min. before the signal for the beginning of the test, which was the ringing of all the bells on the telephone system leading to the different observers. The valve-man then quickly adjusted the valve, by direction of the gauge-reader, until the meniscus of the solution descended in the glass tube at a specified rate, usually about 30 or 40 sec. per calibration division, which corresponds to about 8.46 cu. ft. This usually consumed 2 or 3 min. After this, the valve was adjusted from time to time during the tests so as to maintain, approximately, a uniform rate of discharge.

The gauge-reader then began to note the instants of time when the meniscus passed the successive calibration divisions, the data being entered on the second sheet. One minute before the beginning of the test a preliminary signal was given, and at the beginning there was another signal. The gauge-reader, however, merely draws a line under the last recorded item at the beginning signal, making no other observations. He continues to enter the successive transits of the calibration marks until the signal at the end of the test is sounded,

when he again draws a line under the last recorded item. After taking several more transits, for 1 or 2 min., the flow of salt solution is stopped and the test is at an end.

The adequacy of this proceeding will be seen by noting that there is really no well-defined beginning or end of a turbine test, and that, in fact, the rate of flow of salt solution is substantially the same whether it is determined from the calibration divisions immediately preceding or following the lines drawn when the signals are given for the nominal beginning and end of the test.

The only serious care necessary to take is to see that the rate of flow from the tank is about the same for the first few divisions before and after the beginning, and before and after the end. of the test, so that the time of discharging the standard volume is nearly the same at the first and last divisions, which are indicated on the record by lines drawn under the entries made by the gauge-reader.

If stop-watches are used it will be necessary to run the test for 15 min. in order to limit orders to $\frac{1}{10}$ of 1% with certainty, because, for ordinary field methods, there would be likely to be an error of nearly 1 sec. Stop-watches are not very satisfactory for accurate time observations.

If a chronograph is used, time can be determined with a precision of $\frac{1}{10}$ of 1%, with ordinary care, in 2 or 3 min., or even less. Hence it is clear that, with a given size of tank, a saving of chemical can be effected by the use of a chronograph, where high precision is demanded. as was the case in the tests described herein.

64.—Gravimetric Calibration of Calibrator.—Fig. 5 exhibits the results of a series of calibrations of the iron pipette, or calibrator, which was used in graduating the capacity scale for the gauge-board at the side of the salt-solution tank, as described in Section 62.

It will be seen from Fig. 5 that the three calibrations of the calibrator were made at different times, with a change of observers after the first series. The volumes at 20° cent. computed from the averages of the weighings in the three series are, respectively:

July 13th, 1914.................	8.4661 cu. ft.
October 22d, 1914...............	8.4662 cu. ft.
October 27th, 1914..............	8.4651 cu. ft.

It is possible that this close agreement is principally due to the fact that the weighings were all made on the same scale, which, though for commercial purposes, was carefully adjusted with standard weights

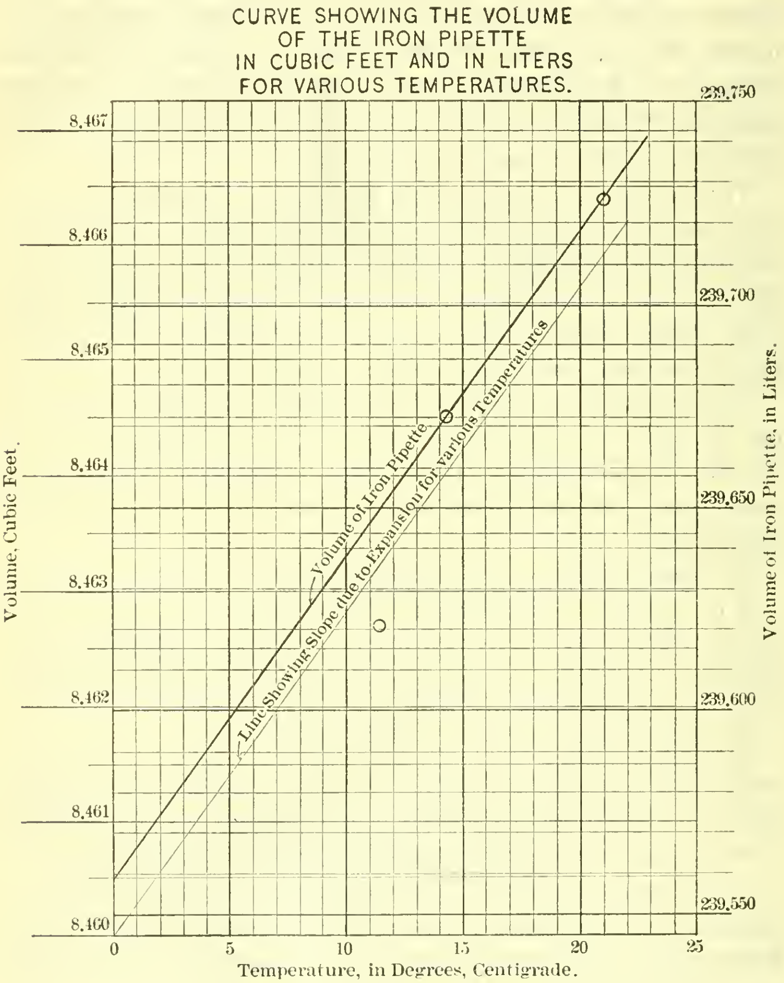


FIG. 5.

prior to each series of weighings. There is no reason to doubt the high precision of these weighings, considered as the results of engineering operations in the field.

TABLE 28.—CALIBRATION OF IRON PIPETTE BY GRAVIMETRIC METHOD.
BY G. H. BANCROFT AND R. E. WARD.

July 13th, 1914. Howe Scale No. 656 849.

No.	Weight, in Pounds.			Temperature of water, in degrees, centigrade.	Time.	Remarks.
	Tank full.	Tank empty.	Weight of water.			
*1........	1 606.75	1 078.00	528.75	21.0	A. M.	Canal water was used.
*2........	1 605.28	1 078.12	527.16	20.8	A. M.	
3........	1 605.75	1 078.37	527.38	20.5	A. M.	
4........	1 605.75	1 078.25	527.50	21.3	P. M.	
5........	1 605.55	1 078.10	527.45	21.0	P. M.	
6........	1 605.40	1 077.87	527.53	21.1	P. M.	
7........	1 605.40	1 078.00	527.40	21.1	P. M.	
8........	1 605.20	1 077.67	527.53	21.0	P. M.	
Mean..	1 605.508	1 078.043	527.465	21.0		

* Readings 1 and 2 omitted in average.

Weight of 1 c.c. of water at 4° cent. = 1 0 grammes. (From Landolt and Börnstein.)
Weight of 1 cu. m. (= 1 000 liters) of water at 4° cent. = 1 000 kg.
1 cu. m. = 35.31658 cu. ft. (Van Nostrand's Chemical Annual.)
1 kg. = 2.20462 lb. (Van Nostrand's Chemical Annual.)

Therefore, weight of 1 cu. ft. of water at 4° cent. $= \frac{1\ 000}{35.31658} = 28.31531$ kg.

or weight of 1 cu. ft. of water at 4° cent. = 28.31531 × 2.20462 = 62.4245 lb.
Weight of 1 c.c. of water at 21° cent. = 0.998019 grammes. (Landolt and Börnstein.)
or weight of 1 cu. ft. of water at 21° cent. = 62.4245 × 0.998019 = 62.301 lb.

Therefore, number of cubic feet in iron pipette at 21° cent. $= \frac{527.465}{62.301} = 8.4664$.

1 cu. ft. = 28.31531 liters.
8.4664 × 28.315 = 239.73 liters in pipette at 21° cent.
Linear coefficient of expansion of iron = 0.0000111 for 1° cent. (Landolt).
Cubical coefficient of expansion of iron = 0.0000333 for 1° cent.
To find volume, in cubic feet, at 20° cent.
Difference in temperature = 21° — 20° = 1°.
1 × 0.0000333 = 0.0000333 cubical coefficient of expansion for 1° cent.
Therefore, volume at 20° = 8.4664 — (8.4664 × 0.0000333) = 8.4661 cu. ft.

In order to facilitate the determination of the volume of the calibrator, or large iron pipette, at various temperatures, the curve (Fig. 5) was prepared, and needs no further explanation.

For the purposes of the tests, the volume of this calibrator will be taken from the curve.

65.—Contents and Delivery of Iron Calibrator.—This large calibrator, or iron pipette, Fig. 21, was constructed on the same principle as the pipettes of the chemists. It consisted of a length of 12-in. pipe, in a vertical position, with a conical top and bottom, so as to admit of as complete a discharge from the bottom as possible while any air within would escape at the top. The top of the upper cone was continued upward with a piece of 2-in. pipe for 18 or 20 in., carrying an ordinary water glass at the side on which was a mark indicating

TABLE 29.—CALIBRATION OF IRON PIPETTE BY GRAVIMETRIC METHOD.
BY J. B. CAMPBELL, C. M. SCUDDER, AND D. C. WALSER.
October 22d, 1914. Howe Scale No. 656 849.

No.	Weight, in Pounds.			Temperature of Water, in Degrees, Centigrade.				Time.	Remarks.
	Tank full.	Tank empty.	Water.	From hose.	Tank one-half full.	Tank full. Top.	Tank full. Bottom.		
1	....	*1 139.3	.....	....	....	....	....	A. M.	Tank empty but dry.
2	1 668.1	1 140.2	527.9	14.3	14.0	14.0	14.0	8:30– 9:30	Canal water was used.
3	1 668.0	1 139.9	528.1	14.0	14.0	14.0	14.0	9:30–10:25	
4	1 667.7	1 139.7	528.0	15.2	14.2	14.0	14.0	10:25–11:00	
Mean	1 667.93	1 139.93	528.0	Mean temperature 14.14° cent.					Room temperature 13.0° cent.

* Reading 1 omitted in average.

Weight of 1 cu. ft. of water at 4° cent. is 62.4245 lb
Weight of 1 c.c. of water at 14.14° cent.=0.9992514 grammes. (Landolt and Börnstein.)
Weight of 1 cu. ft. of water at 14.14° cent.=62.4245 × 0.9992514 = 62.378 lb.

Therefore, number of cubic feet in iron pipette at 14.14° cent. = $\frac{528.0}{62.378}$ = 8.4645.

Number of liters in iron pipette at 14.14° cent. = 8.4645 × 28.315 = 239.67.
To find volume of iron pipette at 20° cent. in cubic feet.
Change of temperature = 20° — 14.14° cent. = 5.86° cent.
Cubical coefficient of expansion for 1° cent. = 0 0000333.
Cubical coefficient of expansion for 5.86° cent. = 0.000195.
Therefore, volume at 20° = 8.4645 + (8.4645 × 0.000195) = 8.4662 cu. ft.

the proper level of the water for a standard volume. The capacity, when filled to this mark, was a little more than 241 liters, and the discharge from it when filled with canal water was very close to 240 liters, or about 1 liter remained in the calibrator when it was emptied through the conical bottom which was provided with a throttle valve.

The large iron calibrator had the characteristic of a chemical flask, or chemical pipette, in that the contents were greater than the delivery. Owing to this difference, the chemical calibration of the contents does not equal the gravimetric calibration of the delivery, but it will be seen that the capacities of the large solution tank, determined chemically and gravimetrically, agree closely, so that by means of the agreement between the results of the two kinds of calibration of the solution tank, the gravimetric and chemical calibrations of the calibrator check each other very satisfactorily, in view of the unfavorable conditions under which the chemical calibrations of both took place. This calibrator, or iron pipette, is more fully described in Part III.

TABLE 30.—Calibration of Iron Pipette by Gravimetric Method.

By J. B. Campbell, C. M. Scudder, and D. C. Walser.

October 27th, 1914. Howe Scale No. 656 849.

No.	Weight, in Pounds.			Temperature of Water, in Degrees, Centigrade.				Time.	Remarks.
	Tank full.	Tank empty.	Water.	From hose.	Tank one-half full.	Tank full. Top.	Tank full. Bottom.		
								A. M.	
1............	....	*1 127.8	....	....	....	....	....	....	Tank empty but dry.
2............	1 656.9	1 128.8	528.1	11.1	10.0	11.3	9.5	9:55–10:40	3.4° cent. temperature of air.
3............	1 656.9	1 129.0	527.9	11.1	10.0	11.2	10.4	10:40–11:17	4.0° cent. temperature of air.
4............	1 656.9	1 129.0	527.9	11.1	11.0	11.1	11.0	11:17–11:50	
								P. M.	
5............	1 656.9	1 128.6	528.3	11.1	10.5	12.5	10.0	1:40 – 2:20	4.8° cent. temperature of air.
6............	1 657.0	1 128.9	528.1	12.5	12.5	12.5	12.2	2:20 – 3:00	6.2° cent. temperature of air.
7............	1 657.0	1 129.0	528.0	12.0	12.0	12.0	11.0	3:00 – 3:35	
8............	1 657.4	1 129.3	528.1	11.8	12.0	12.0	12.0	3:37 – 4:15	7.6° cent. temperature of air.
9............	1 657.2	1 129.1	528.1	....	12.0	12.0	11.6	4:15 – 4:45	
Mean......	1 657.025	1 128.9625	528.0625	11.53	11.25	11.83	10.96		Canal water was used.
				Mean temperature = 11.4° cent.					

* Reading 1 omitted in average.

Weight of 1 cu. ft. of water at 4° cent. is 62.4245 lb.
Weight of 1 c.c. of water at 11.4° cent. = 0.999591 grammes (Landolt and Börnstein).
Weight of 1 cu. ft. of water at 11.4° cent. = 62.4245 × 0.999591 = 62.399 lb.

Therefore, number of cubic feet in iron pipette at 11.4° cent. = $\frac{528.0625}{62.399}$ = 8.4627.

Number of liters in iron pipette at 11.4° cent. = 8.4627 × 28.315 = 239.62.
To find volume of iron pipette at 20° cent. in cubic feet.
  Change of temperature = 20° — 11.4° = 8.6° cent.
  Cubical coefficient of expansion for 1° cent. change = 0.0000333.
  Cubical coefficient of expansion for 8.6° cent. = 0.000286.
Therefore, volume at 20° cent. = 8.4627 + (8.4627 × 0.000286) = 8.4651 cu. ft.

66.—Chemical Determination of Capacities.—The chemical calibrations of the solution tank and the iron calibrator are given in Tables 32-38 and 41, Sections 66, 67, and 68. In order to illustrate the methods more fully, the titrations of all the samples in the case of the calibrator are given in Tables 32 and 33. These tables also show the relative accuracy of the chemical work. In particular, attention must be directed to the fact that two of these calibrations are less trustworthy than the remainder, and that all were executed under more adverse conditions than the regular tests of the turbines. Those in October were mostly performed in a drizzling rain, at the close of a long series of regular tests, and after some of the chemists had returned to their duties at various colleges. Those of November were performed by an entirely different corps of chemists, and during very cold weather, with the atmospheric temperature in the neighborhood of zero, Fahrenheit. The writer supervised the measurement of the 21½ liters of salt solution for the lower level of the tank on November 24th, when its temperature was found to be — 2° cent. This was done in the laboratory where the temperature was nearly 20° cent. higher. If such unfavorable conditions could have been avoided, there is no doubt that a precision surpassing that of the best of the turbine tests would have been secured. As the tests stand, however, they are probably more valuable to the engineer as examples of what can be accomplished under adverse conditions, such as are experienced in the field, than they would have been under more perfect laboratory conditions.

It has been stated that two of the chemical calibrations are not as trustworthy as the others. These are the tests of November 23d, at the upper level of the solution tank, and those of November 25th on the iron calibrator. The first of these was rendered untrustworthy by a failure to measure accurately the quantity of salt solution introduced into the tank from the calibrator. There were, however, nine samples, averaging about 200 c.c. each, taken for this purpose, so that 1 800 c.c. have been deducted from the capacity of the calibrator to ascertain the value of q for this test. In the case of the calibration of the iron calibrator on November 25th, the laboratory work does not check with sufficient exactness to support confidence in the result, as shown by the calculations in Table 31.

The titrations for the upper and lower levels of the large tank show that there was relatively considerable salt still remaining in the tank

TABLE 31.—CALCULATIONS OF CONCENTRATION OF NORMAL HEAD-WATER SAMPLES TAKEN FROM LARGE TANK AND CALIBRATOR BEFORE INTRODUCING SALT SOLUTION IN THE CHEMICAL CALIBRATIONS.

Symbol.	Calibrator.		Tank.			
			Upper level.		Lower level.	
	October 14th.	November 25th.	October 15th.	November 23d.	October 16th.	November 24th.
R'	100.65	*100.97	100.48	100.59	100.51	100.64
c'_2	5 490.9	5 326.9	5 757.1	5 454.0	5 513.6	5 899.5
c	550 295	535 030	558 091	530 851	523 936	533 814
r'	99.68	100.00	99.51	99.61	99.53	99.66
$R\,c'_2$	552 659	537 857	578 473	548 618	554 172	593 726
c_1	23.72	28.27	204.82	178.37	303.79	601.16

* Interpolated value from following calculations.

after it had been thoroughly cleaned and prepared for the chemical calibration, and that this fact is more apparent at the lower level, as might be expected.

By Equation (49), Section 21, the titration of normal head-water, November 25th, is found, by using the data of Table 34 (Plate LI), to be

$$v_1 = \frac{100.82 \times 5\,326.9 - 535\,030}{99.85} = 20.31,$$

whereas the result should have been about 28 c.c., as may be seen by examining the titrations of the normal head-water on the turbine tests, either observed or computed, see Part IV, or Table 7, Section 46, where it will be found that the titration of the normal head-water was about 26 c.c., as an average, when the strength of the silver nitrate was 1.562, and about 28 c.c. when the strength was 1.45, which is in perfect accord with theory, as

$$\frac{1.562}{1.45} \times 26 = 28.$$

Notice that v_1 for the test of October 14th may be computed to be 23.7 c.c., which is within about 2 c.c. of what it should be, and thus sufficiently accurate.

In order to correct this error for November 25th, slight as the effect is, the value of R' has been computed by using Equation (99), substituting about 28 c.c. for c_1, so that

$$R' = \frac{535\,000}{5\,299} = 100.96$$

Accordingly, this value (more correctly computed = 100.97) of R' has been arbitrarily substituted at the proper place in Table 35, thus giving a somewhat better estimate of the volume. The result, however, is still somewhat doubtful, as the samples of salt solution and other liquids involved do not run as uniformly as they should. Individual titrations of the same solution should agree within about 0.1 c.c., and averages of from five to ten such titrations should check similar averages within $\frac{1}{20}$ of 1% when the total titration of any sample is about 50 c.c. Thus the chemical work of November, as a whole, will not be found so good as that of the earlier tests. These errors, however, are more accidental than systematic, so that curves plotted from numerous tests should not be seriously in error, theoretically being of the same precision as the earlier curves, if there are correspondingly more observations.

To gain a better view of the chemical work, the titrations of the chemical gaugings of the capacity of the iron calibrator are given in Tables 32 and 33.

TABLE 32.—Titrations for Chemical Determination of Volume of Iron Calibrator.

October 14th, 1914.

	Salt solution. 1/100 c.c		Special dilutions. 1/100 c.c.		Tail-water. 1/100 c.c.	
	5 485	5 480	....	....	5 500	5 515
	5 475	5 470	5 500	5 500	5 510	5 500
	5 460	5 458	5 500	5 495	5 505	5 505
	5 480	5 484	....	....	....	....
	5 480	5 477	....	....	....	....
Means............	5 476	5 473.8	5 500	5 497.5	5 505	5 506.7
Average..........	5 474.9		5 498.8		5 505.8	

October 15th, 1914.

	Salt solution. 1/100 c.c		Special dilutions. 1/100 c.c.		Tail-water. 1/100 c.c.	
	5 470	5 480	5 490	5 485	5 495	5 495
	5 470	5 460	5 485	5 490	5 495	5 495
	5 465	5 460	5 485	5 490	....	....
Means............	5 468.3	5 466.7	5 486.7	5 488.3	5 495	5 495
Average..........	5 467.5		5 487.5		5 495.0	
Final average....	5 471.2		5 493.1		5 500.4	

TABLE 33.—TITRATIONS FOR CHEMICAL DETERMINATION OF VOLUME OF IRON CALIBRATOR, NOVEMBER 25TH, 1914.

Titrated November 26th, 1914.

Salt solution, 1/100 c.c.			Special dilutions, 1/100 c.c.			Tail-water, 1/100 c.c.		
1	5 292	5 300	1	5 300	5 306	*B.*	(5 365)	(5 372)
2	5 325	5 325	2	5 332	5 332	*T.*	(5 305)	(5 310)
3	5 272	5 276	3	5 290	5 295	*B.*	(5 359)	(5 360)
4	5 326	5 317	4	5 340	5 344	*T.*	5 335	5 333
			3	(5 260)	(5 263)	*B.*	5 339	5 323
			4	5 343	5 345	*T.*	5 337	5 335
Means ..	5 303.8	5 304.5		5 321	5 324.4		5 337	5 330.3
Average	5 304.1			5 322.7			5 333.7	

In Table 33 the samples are given in the order taken, those marked *B.* coming from the sampling cock at the bottom of the calibrator and those marked *T.* from the top. Thus, it is seen at once that the first three samples of tail-water, taken while the mixture was being stirred, indicate that a perfect mixture had not then been secured, the heavy salt solution naturally settling toward the bottom. The samples were taken from 3 to 5 min. apart, and each has been titrated twice in the laboratory, as may be detected by examining the titrations in pairs, which are on the same line. Therefore the tail-water samples in parentheses have been omitted from the averages of the titrations. In the case of the special dilutions, the prefixed numerals relate to the salt solution sample with which the normal head-water was mixed. The second set of these, which was made from Sample 3, titrated less than the dilute salt solution, which is inconsistent. In fact, all these samples titrate too low, as they should run, on the average, about 0.28 c.c. higher than the salt solution titration, which latter averages 5 304, or they should average about 5 332, instead of 5 322.7 as they do. This discrepancy has already been discussed in determining a more probable value of R'. For these reasons, the titrations 5 260 and 5 263 have been excluded from the average, and the last set, though throwing too much weight in favor of No. 4 sample, is allowed to be included, as it is clear that the titrations of special dilutions are all too low.

Comparing now the results of the titrations in Tables 32 and 33, it will be easy to see that the former is far the more trustworthy, and for this reason a second computation of the volume of the salt-solution tank in the tests of October and November has been made, using the

calibration of the calibrator of October 14th only. These latter calculations are undoubtedly the more correct, especially that for November 23d, because there is no question as to the quantity of salt solution introduced into the salt-solution tank in that test.

The detailed calculations appear in Tables 34 (Plate LI), and 35. In compiling Table 34 the volumes of samples of tail-water and special dilutions have been supposed to vary with changes in temperature in the same ratio as the specific volume of distilled water. Although this is sufficiently accurate in all cases, yet, in computing the turbine tests, it was decided to take the actual densities of tail-water and special dilutions as closely as they could be found, by using Table 1 and Plates XLVIII, XLIX, and L. The tabular densities of the salt solution have been utilized in the same way, and, as the ratios, only, of these densities are used in the calculations, no serious errors result, even where the true densities differ appreciably from the tabular values, due to the fact that commercial salt was used instead of pure NaCl.

We are now in a position to compare the best results of the chemical calibrations of the salt solution tank with the results of the calibrations based on the gravimetric determination of the volume of the calibrator in Section 64.

Plate LII shows the graduation marks on the gauge-board at the side of the solution tank. Each calibration division represents a calibrator-full of water, as it was drawn from the tank, the graduation marks having been placed on the scale by using a lead pencil and carpenter's square at the level of the meniscus, which was plainly visible in the 1-in. glass tube in the slot along the center of the board.

There were three calibrations on different dates, as follows:

1.—Long marks, July 16th, 17th, 18th, 20th, and 21st, by Messrs. Ward and Bancroft;

2.—Short marks, July 30th, 31st, and August 1st, by Messrs. Ward and Bancroft;

3.—Intermediate marks, October 20th, by Messrs. Ward and Campbell.

In the chemical calibration of October, the upper level of the water in the tank was at graduation mark 12 of the short marks before the salt solution (1 calibrator-full, less about 1 800 c.c. for samples) was introduced, thus filling the tank nearly to graduation 11. At the lower level the water surface was at graduation 70 of the short marks.

TABLE 34.—Data and Preliminary Computations for the Chemical Calibrations of Iron Calibrator and Salt Solution Tank in the Tests of October 14th, 15th, 16th, and November 23d, 24th and 25th, 1914.

No. of line.			Formulas.	Symbol.	Iron Calibrator.		Salt-Solution Tank.			
							Upper level.		Lower level.	
					October 14th.	November 25th.	October 15th.	November 23d.	October 16th.	November 24th.
1	Temperatures:	Salt solution when measured for tank		....	17.0	16.4	10.0	1.0	13.9	—2.0
2		" " " pipetted for dilution with distilled water		....	17.0	19.4	10.0	11.88	14.0	18.4
3		" " " " " special dilutions		....	17.0	19.4	10.0	11.88	14.0	18.8
4		Distilled water for dilution of salt solution		....	18.75	20.0	19.0	14.75	17.25	18.0
5		Head-water for special dilutions		....	15.0	16.1	14.0	11.15	15.0	14.84
6		Pipetting mixture in laboratory		....	14.0	14.5	14.5	12.0	15.3	14.0
7		Mixture in tank		....	13.9	4.0	14.2	6.0	15.9	7.5
8		Head-water for diluting mixture in laboratory		....	....	....	14.75	11.15	....	....
9		" " in tank before introducing salt		....	13.9	4.0	14.0	5.0	15.3	8.5
	Volumes				No. 18 791	Nos. 9 206 and 9 171	No. 18 791	Nos. 9 206 and 9 171	No. 18 791	Nos. 9 206 and 9 171
10		Salt solution discharged by pipette for dilute salt solution at temperature of Line 2		....	9.976	9.981	9.969	9.974	9.974	9.980
					Nos. 1 456 and 1 476	Nos. 1 407 and 1 457	Nos. 1 474 and 1 456	Nos. 1 407 and 1 457	Nos. 1 474 and 1 456	Nos. 1 457 and 1 407
11		Flask measuring distilled water at temperature of Line 4		....	1 003.14	1 001.8	1 003.2	1 001.2	1 003.2	1 001.8
					No. 21 047	Nos. 9 206 and 9 171	No. 21 047	Nos. 9 206 and 9 171	No. 21 047	Nos. 9 206 and 9 171
12		Diluted salt solution discharged by pipette at temperature of Line 4		....	10.011	10.006	10.011	10.000	10.009	10.004
13		Titration per pipette of diluted salt solution sample in laboratory		....	54.712	53.041	55.604	62.747	52.175	52.999
14		Titration of diluted salt solution per liter in laboratory	Line 13 ÷ Line 12	....	5 465.2	5 300.9	5 554.3	6 274.7	6 212.8	5 297.8
15		Ratio of dilution of diluted salt solution with distilled water	Line 11 ÷ Line 10	....	100.56	100.32	100.63	100.38	100.58	100.83
16		Titration per liter of salt solution at temperature of Line 2	Line 14 × Line 15	....	549 681	631 786	558 929	529 474	624 303	531 528
17		Concentration (approximate) in grammes per liter of salt solution at temperature of Line 2		....	295.43	265.89	300.45	264.74	281.84	265.76
18		Percentage of salt in solution		....	24.86	22.72	25.2	22.60	23.85	22.69
19	Densities	Salt solution when measured at temperature of Line 1		....	1.1881	1.1710	1.1940	1.1768	1.1813	1.1791
20		" " " pipetted for dilution at temperature of Line 2		....	1.1881	1.1593	1.1940	1.1721	1.1812	1.1699
21		" " " " " special dilutions at temperature of Line 3		....	1.1881	1.1693	1.1940	1.1721	1.1812	1.1699
22		" " at temperature of mixture in tank, temperature of Line 7		....	1.1897	1.1764	1.1922	1.1751	1.1804	1.1749
23		Titration of salt solution per liter when measured for tank, temperature of Line 1	Line 16 × Line 19 ÷ Line 20	....	549 581	532 584	558 929	531 592	524 355	535 727
24		" " " " " " at temperature of mixture in tank, temperature of Line 7	Line 16 × Line 22 ÷ Line 20	v	550 295	535 030	558 091	530 851	523 936	533 814
	Volumes......				Nos. 1 474, 1 456, 11—100 c.c.	Nos. 1 457, 1 407, 3—100 c.c.			No. 1 476	Nos. 1 457 and 1 407
25		Salt solution when measured for tank at temperature of Line 1		....	2 406.9	2 402.8	239 530	240 770	20 067	21 516
26		" " for tank at temperature of mixture in tank, temperature of Line 7	Line 25 × Line 19 ÷ Line 22	q	2 403.8	2 391.8	239 889	241 131	20 083	21 594
					No. 18 791	Nos. 9 205 and 9 171	No. 18 791	Nos. 9 206 and 9 171	No. 18 791	Nos. 9 206 and 9 171
27		" " discharged by pipette for special dilutions, temperature of Line 3		....	9.976	9.981	9.969	9.974	9.974	9.980
28		Distilled water at temperature of mixture in tank, temperature of Line 7		....	1.0007	1.0000	1.0008	1.0000	1.0010	1.0001
29		" " " " " special dilutions " " " 6		....	1.0009	1.0010	1.0007	1.0004	1.0009	1.0008
30		" " " " " pipetting mixture in laboratory, temperature of Line 6		....	1.0007	1.0008	1.0008	1.0005	1.0009	1.0007
					Nos. 1 456 and 1 476	Nos. 1 407 and 1 457	Nos. 1 474 and 1 456	Nos. 1 407 and 1 457	Nos. 1 474 and 1 456	Nos. 1 457 and 1 407
31		Flask for special dilutions at temperature of Line 5		....	1 003.05	1 001.2	1 003.1	1 001.1	1 003.1	1 001.2
32		Special dilutions at temperature of mixture in tank, temperature of Line 7	Line 31 × Line 28 ÷ Line 29	Q'	1 002.8	1 000.2	1 003.2	1 000.7	1 003.9	1 000.5
33		Salt solution for special dilutions at temperature of mixture in tank, Line 7	Line 27 × Line 21 ÷ Line 22	q'	9.963	9.921	9.984	9.948	9.981	9.937
34		Ratio of dilution for special dilutions	Line 32 ÷ Line 33	R'	100.65	100.82*	100.48	100.59	100.51	100.64
35		Value of k or k, approximate		k'	0.9744	0.9746	0.9737	0.9755	0.9767	0.9753
36		Value of k' or k' "		k	0.9744	0.9746	0.9737	0.9755	0.9767	0.9753
					No. 21 047	Nos. 9 205 and 9 171	No. 21 047	Nos. 9 206 and 9 171	No. 21 047	Nos. 9 206 and 9 171
37		Volume of pipette for special dilutions, temperature of Line 5		....	10.006	10.002	10.006	9.997	10.008	10.001
38		Titration per pipette of special dilutions		....	54.931	53.227	57.606	54.502	55.174	58.960
39		Titration per liter of special dilutions at temperature of Line 5	Line 38 ÷ Line 37	....	5 489.8	5 321.6	5 757.7	5 451.8	5 514.1	5 895.4
40		Titration of special dilutions at temperature of mixture, Line 7	Line 39 × Line 29 ÷ Line 28	v'_2	5 490.9	5 326.9	5 757.1	5 454.0	5 513.6	5 899.5
					No. 21 047	Nos. 9 206 and 9 171	No. 21 047	Nos. 9 206 and 9 171	No. 21 047	Nos. 9 206 and 9 171
41		Volume of pipette measuring mixture from tank, temperature of Line 6		....	10.005	10.001	10.005	9.998	10.006	10.000
42		Titration per pipette of Line 41		....	55.004	53.337	56.808	55.216	51.607	59.591
43		Titration per liter of mixture at temperature of Line 6	Line 42 ÷ Line 41	....	5 497.7	5 333.2	5 678.0	5 522.6	5 157.6	5 959.1
44		" " " " " " " mixture, Line 7	Line 43 × Line 30 ÷ Line 28	v_2	5 497.7	5 337.5	5 678.0	5 525.4	5 157.1	5 962.7

* This ratio not being in reasonable agreement with the concentrations of salt solution and special dilutions, is probably in error. Based upon the concentrations, the ratio should be 100.97, which therefore has been adopted for this calibration.

TABLE 35.—FINAL COMPUTATIONS FOR THE CHEMICAL CALIBRATIONS OF IRON CALIBRATOR AND SALT SOLUTION TANK IN THE TESTS OF OCTOBER 14TH, 15TH, 16TH, AND NOVEMBER 23D, 24TH, 25TH, 1914.

Line.		Iron Calibrator.		Salt Solution Tank.			
				Upper level.		Lower level.	
		Oct. 14.	Nov. 25.	Oct. 15.	Nov. 23.	Oct. 16.	Nov. 24.
1	k''	0.976	0.975	0.972	0.978	0.976	0.979
2	c_2	5 498.00	5 337.50	5 678.0	5 525.4	5 157.1	5 962.7
3	(See Equations 106) c'_2	5 490.90	5 326.90	5 757.1	5 454.0	5 513.6	5 899.5
4	$k'' c_2$	5 366.048	5 204.06	5 519.0	5 403.8	5 033.3	5 837.5
5	c	550 295.0	535 030.00	558 091.	530 851.	523 936.	533 814.
6	$c - k'' c_2$	544 928.95	529 825.9375	552 600.	525 000.	518 900.	527 976.
7	$c_2 - c'_2$	7.100	10.600	−79.1	71.4	−356.5	63.2
8	$f = \left(c_2 - c'_2 \right) \div \left(c - k'' c_2 \right)$....	0.000013029	0.0000200	−0.0001431	0.000136	−0.0006870	0.000120
9	R'	100.65	100.97	100.48	100.59	100.51	100.64
10	$R'f$	0.00131	0.00202	−0.01438	0.0137	−0.06905	0.0121
11	$r' = R' - k'$	99.68	100.00	99.508	99.612	99.534	99.661
12	$r = r' \div (1 + R'f)$	99.55	99.80	100.96	98.266	106.92	98.470
13	k	0.974	0.97	....	....	0.976	0.979
14	$R = r + k$	100.524	100.77	....	....	107.90	99.449
15	q	2 403.8	2 391.8	....	....	20.083	21.594
16	$Rq = Q$ at temperature of mixture in tank.	241.64 @ 13.9° cent.	241.02 @ 4° cent.	....	....	2 167.0	2 147.5
	Correction	....	−0.04	....	....	....	....
17	Corrected..............................	....	240.98 "	....	....	....	....
18	Volume of calibrator at 13.9° C...........	241.64	241.06	....	....	....	....

TABLE 35.—(*Continued*).

Line.		Salt Solution Tank. Upper level. Oct. 15.	Upper level. Nov. 23.	Lower level. Oct. 16.	Lower level. Nov. 24.
	Calculation of r'', r_x, and R_x.				
19	Temperature of head-water for diluting mixture in laboratory	14.75	11.15		
20	" " mixture from tank for dilution	14.	12.		
21	" " mixture in tank	14.2	5.		
22	Volume of ½ liter flask at temperature of Line 19	500.53	500.24		
23	" " 1 " " " " " " 20	1 003.1	1 001.1		
24	Density of normal head-water at temperature of Line 19	999.4	999.4		
25	" " " " " " " " " 21	999.5	1 000.0		
26	" " mixture at temperature of Line 20	1 001.3	1 002.8		
27	" " " " " " " 21	1 001.3	1 003.2		
28	Vol. of N. H. water for special dilutions at temperature L. 21 = L. 22 × L. 24 ÷ L. 25	500.48	499.94		
29	Vol. of mixture for special dilutions at temperature L. 21 = L. 23 × L. 26 ÷ L. 27	1 003.1	1 000.7		
30	r'' L. 28 ÷ L. 29	0.4989	0.4996		
31	k_x	0.974	0.973		
32	$r_x = (r - r'' k_x) \div (1 + r'')$ See Equations 83.	67.032	....		
33	$R_x = (r + k_x) \div (1 + r'')$ See Equations 83.	....	66.177		
34	q = mean of two iron calibrator volumes, Line 18 at 13.9° C.	241.35	241.35		
35	q reduced to temperature of salt solution when measured for tank	239.53 at 10° cent.	240.77 at 1° cent.		
36	q " " " " mixture in tank. (See Lines 25 and 26, Table 31.)	239.89 at 14.2° cent.	241.13 at 5° cent.		
37	$Q = r_x\, q$ and $R_x\, q$, respectively, based on vol. iron calibrator, October 14 and November 25	16 081.	15 957.		
38	q = vol. of calibrator as found on October 14. (See Line 18)	241.64 at 13.9° cent.	241.64 at 13.9° cent.		
39	q after deducting 1.8 and 0.48 for samples, respectively, and reducing to temperature of salt solution when measured for tank	239.82 at 10° cent.	241.06 at 1° cent.		
40	q reduced to temperature of mixture in tank, Line 39 × $\frac{\text{Line 19}}{\text{Line 22}}$ (Table 34)	240.18 at 14.2° cent	241.40 at 5° cent.		
41	$Q = r_x\, q$ and $R_x\, q$, respectively, based on vol. iron calibrator, October 14 only.	16 100.	15 975.		

In the chemical calibration of November, at the upper level, the water surface was at the point marked *A*, Plate LII, slightly below graduation 13 of the long marks; and, at the lower level, the water surface was at graduation 70 of the long marks.

Therefore, to compare the two chemical calibrations, it is necessary to reduce them to the same levels, which, for definiteness, will be at graduations 12 and 70 of the long marks. These corrections, therefore, are due to discrepancies of 0.17 in. (scaled from Plate LII) between the long and short marks for graduation 70, and 0.03 in. between the long and short marks for graduation 12, the average lengths of division for 1 calibrator-full at these levels being, respectively, 1.41 and 1.49 in., giving percentage corrections of $17 \div 1.41 = 12\%$ and $3 \div 1.49 = 2\%$, correspondingly. The capacity of a calibrator being approximately 240 liters, the corrections in liters are:

For upper level = 2% of 240 = 5 liters,
For lower level = 12% of 240 = 29 liters.

In both cases these corrections should be deducted from the volumes given by the chemical calibration of October, to reduce to the proper graduation mark.

In the case of the chemical calibration of November, a correction must be added to the volume at the upper level to obtain the corresponding volume at the long mark for division 12. The correction to reduce from the mark, *B*, to long graduation mark 12 is due to a discrepancy of level of 0.3 in. in an average length of 1.50 in., or exactly 20% of 1 calibrator-full, which amounts to 48 liters to be added to the November volume at the upper level.

Table 36 gives the adjustment of the figures from Table 35.

It must be remembered that the November calibration here is the more truthworthy, being 13 875 liters between graduation marks 12 and 70.

To obtain figures for comparison from the gravimetric calibrations of the calibrator and the calibration scales of Plate LII, an account must be made of the temperatures at which the scales on the drawing were made. The calibration at long marks in July was made at an average temperature of $20\frac{1}{2}°$ cent.; and those in October, at the intermediate marks, were at $13\frac{1}{4}°$ cent. The short marks will be neglected at present for the reason that there is some doubt about them, as will

appear in the next section. The capacity of the calibrator in this case is about the same as in the case of the long marks, the temperature being 19¾° cent. Thus

Volume of calibrator by Fig. 5 = 239.72 liters at 20½° cent.
= 239.66 liters at 13¼° cent.,

from which

(July 18th) 58 calibrator-fulls at 20½° = 13 904 liters.
(October 20th) 58 calibrator-fulls at 13¼° = 13 900 liters.

TABLE 36.—Adjustment of Volumes Determined Chemically for Comparison with Volumes Determined with the Calibrator and Its Own Gravimetric Calibration.

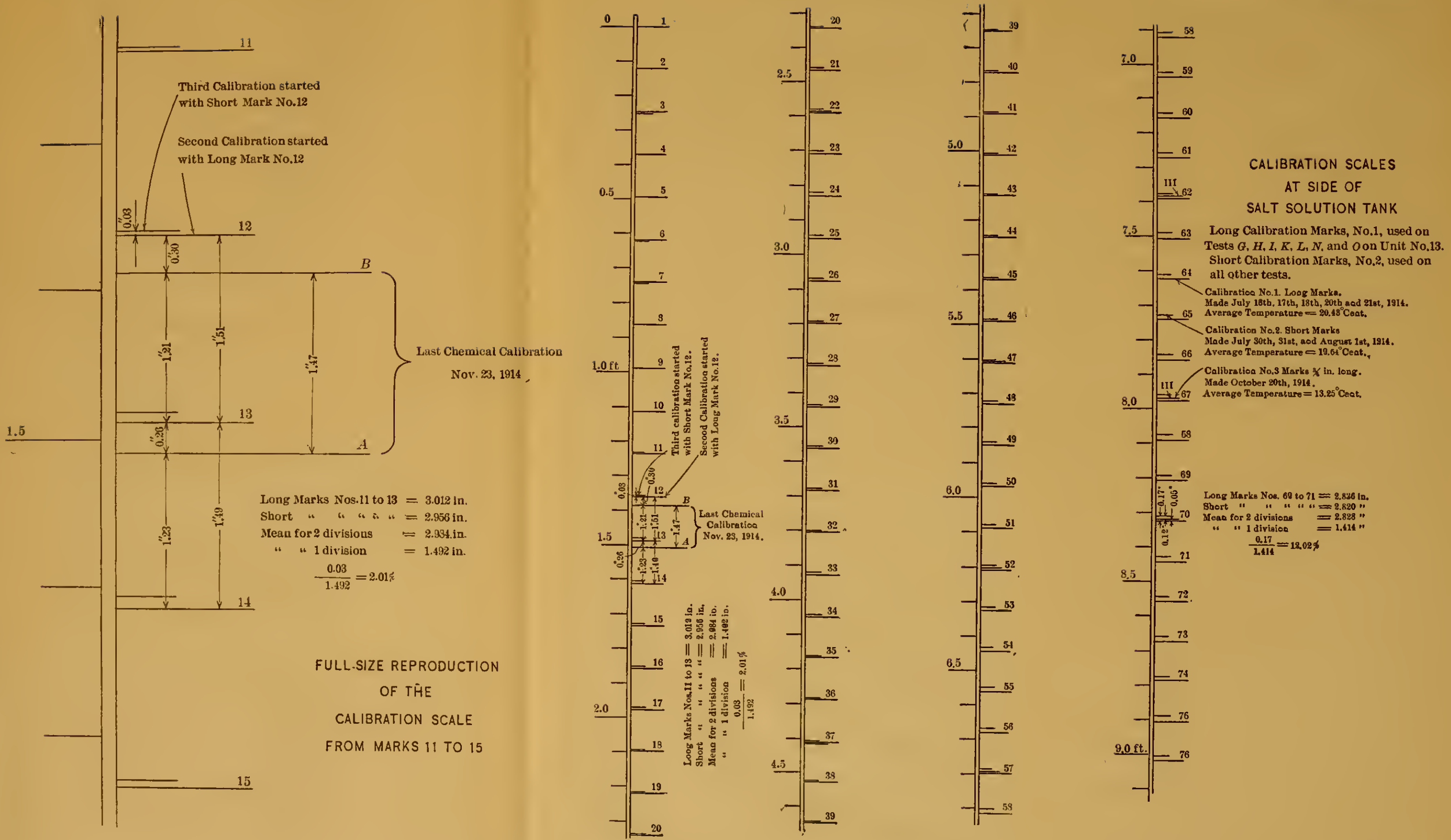
	October 15th.	November 23d.
Volume at upper level, Table 35 (based on volume of calibrator, October 14th only)	16 100*	15 975
Corrections from above	— 05	+ 48
Resulting volume for long mark 12	16 095	16 023
Volume at lower level, Table 35	2 167	2 148
Correction from above	29	......
Resulting volume for long mark 70	2 138†	2 148†
Net volume between long marks 12 and 70, 58 calibrator-fulls	13 957	13 875

* The reader is reminded that there was no exact measurement of the quantity of salt solution taken for samples from the contents of the iron calibrator while it was being emptied into the salt-solution tank on October 15th.

† An error of 0.004 or 0.005 ft. in reading the tank gauge would introduce an error of 8 or 10 liters in the volume computation.

Now, the third calibration (intermediate marks) started with short mark 12 and ended about 0.05 in., or 5 ÷ 1.41 = 3½%, of 1 calibrator-full, 8½ liters, above graduation mark 70. That is to say, the third calibration indicates that the volume from long mark 12 to long mark 70 at the time of the third calibration was 13 900 + 8½ — 5 = 13 903½ liters, or the two calibrations (first and third) check within ½ liter, 58 calibrator-fulls in July, therefore, being 13 904 liters, which is what the volume of the tank from long mark 12 to long mark 70 was in July by the calibration at that time. Hence, the following comparison results:

Volume of tank between graduation marks 12 and 70, of the first calibration by the calibrator, based on both the first and third calibrations	13 904 liters
Volume of tank between these same marks (October)	13 957 "
But by the chemical method (November)	13 875 "
Mean of last two	13 916 "


Third Calibration started with Short Mark No.12
Second Calibration started with Long Mark No.12
Last Chemical Calibration Nov. 23, 1914
Long Marks Nos.11 to 13 = 3.012 in.
Short " " " " " = 2.956 in.
Mean for 2 divisions = 2.934 in.
" " 1 division = 1.492 in.
0.03 / 1.492 = 2.01%
FULL-SIZE REPRODUCTION OF THE CALIBRATION SCALE FROM MARKS 11 TO 15
CALIBRATION SCALES AT SIDE OF SALT SOLUTION TANK
Long Calibration Marks, No.1, used on Tests G, H, I, K, L, N, and O on Unit No.13.
Short Calibration Marks, No.2, used on all other tests.
Calibration No.1. Long Marks. Made July 16th, 17th, 18th, 20th and 21st, 1914. Average Temperature = 20.48°Cent.
Calibration No.2. Short Marks Made July 30th, 31st, and August 1st, 1914. Average Temperature = 19.64°Cent.
Calibration No.3 Marks ⅜ in. long. Made October 20th, 1914. Average Temperature = 13.25°Cent.
Long Marks Nos. 69 to 71 = 2.836 in.
Short " " " " " = 2.820 "
Mean for 2 divisions = 2.828 "
" " 1 division = 1.414 "
0.17 / 1.414 = 12.02%

Apparently, therefore, there was no change in volume of the 58 divisions of the tank by the first and third gravimetric calibrations, but there is considerable change, 82 liters in 13 900, or 0.6%, by the two chemical calibrations, the tank being smaller in November than in October. Attention has been called to the fact that the chemical calibration at the upper level in October is not so trustworthy as the later one, on account of the failure to measure exactly the quantity of salt solution taken from the calibrator for samples. If we neglect this, we may say that the chemical determination indicates a volume of 13 875 liters in November, as against 13 904 liters in July, by means of the calibrator, or only 0.2% discrepancy, and this is in the direction which one might suppose such a change would be.

67.—*Degree of Precision in Turbine Tests.*—Attention has been called to the fact that the chemical calibrations were not conducted under such favorable conditions as the best of the turbine tests. Accordingly, these discrepancies of 0.2 to 0.6% are larger than the discrepancies of the best turbine tests by a considerable margin. However, there is one important conclusion to be drawn from these chemical calibrations, namely: There was no serious systematic error involved in the chemistry and laboratory operations during the turbine test. In so far as the chemical procedure was concerned, the results of the turbine tests are correct to a small fraction of 1 per cent.

It may also be added that the mean of the two chemical tests checks the mean of the first and third calibrations by the calibrator within 0.1 of 1 per cent. If the second calibration with the calibrator is included, the error is still less, but no importance can be attached to these facts. The calculation, however, follows:

Fifty-eight calibrator-fulls in the second calibration (short marks) at $19\frac{3}{4}°$ cent. are equivalent to $58 \times 239.716 = 13\,903\frac{1}{2}$ liters. To reduce this to long marks 12 and 70, simply add, algebraically, the corrections due to the discrepancies in elevation of the corresponding short marks, which have already been ascertained, namely, 29 and 5 liters, respectively. Thus the volume between the long marks 12 and 70, based on the second calibration with the calibrator is $13\,903\frac{1}{2} + 29 - 5 = 13\,928$ liters.

The mean of the values of the volume between divisions 12 and 70, based on all three calibrations with the calibrator, therefore, is the mean of 13 904, 13 904, and 13 928, which is 13 912 liters, as com-

pared with a mean of 13 916 liters by the two chemical calibrations. This omits entirely the last chemical calibration of the contents of the calibrator because of the inconsistencies in the chemical treatment previously mentioned.

We might make other comparisons by various combinations of results, but these would not improve matters, even if they gave closer agreement between the results by the two methods. The writer prefers to adopt the values which can be least criticised. These are based on the calibrator volume determined on October 14th, the tank calibration at the upper level, on November 23d, and the mean of the two calibrations at the lower level, on October 16th and November 24th, both of which appear to be trustworthy. Neglecting the effects of differences of temperature, this estimate would be:

Volume at upper level, November 23d..........	16 023 liters
Mean at lower level October 15th, November 23d. .	2 143 "
Probable volume, 58 divisions.................	13 880 liters.

This agrees well with the gravimetric calibrations of either July or October, the mean of which is 13 902.

To make matters still clearer, Table 37 gives a compilation of the titrations of the samples taken during the chemical calibrations of the large tank.

TABLE 37.—Comparison of Titrations for Calibrations of Large Tank at the Upper Level on October 15th and November 23d.

(All Units in Hundredths of a Cubic Centimeter.)

	Dilute Salt Solutions.				Special Dilutions.				Tail-Water Samples.			
	October 15th.		November 23d.		October 15th.		November 23d.		October 15th.		November 23d.	
	5 555	5 550	5 276	5 265	5 755	5 755	5 433	5 430	5 680	5 675	5 529	5 520
	5 568	5 560	5 250	5 250	5 760	5 765	5 479	5 480	5 685	5 682	5 525	5 527
	5 560	5 565	5 299	5 304	5 770	5 770	5 430	5 433	5 675	5 680	5 518	5 515
	5 565	......	5 251	5 250	5 755	5 750	5 440	5 430	5 680	5 685	5 535	5 535
	......	......	5 279	5 282	5 765	......	5 471	5 457	5 685	......	5 517	5 512
	......	......	5 300	5 290	......	......	5 465	5 455	......	......	5 510	5 515
Means....	5 562	5 558.3	5 275.8	5 273.5	5 761.2	5 760	5 453	5 447.5	5 681	5 680.5	5 522.3	5 520.7
Average..	5 560.4		5 274.7		5 760.6		5 450.2		5 680.8		5 521.5	

TABLE 38.—COMPARISON OF TITRATIONS FOR CALIBRATIONS OF LARGE TANK AT THE LOWER LEVEL ON OCTOBER 16TH AND NOVEMBER 24TH.
(All Units in Hundredths of a Cubic Centimeter.)

	DILUTE SALT SOLUTIONS.				SPECIAL DILUTIONS.				TAIL-WATER SAMPLES.			
	October 16th.		November 24th.		October 16th.		November 24th.		October 16th.		November 24th.	
	5 210	5 215	5 302	5 298	5 520	5 518	5 897	5 895	5 157	5 165	5 927	5 930
	5 220	5 220	5 300	5 301	5 515	5 515	5 905	5 900	5 160	5 162	6 004	5 973
	5 220	5 220	5 300	5 303	5 520	5 523	5 901	5 900	5 160	5 160	5 945	5 945
	......	......	5 300	5 295	5 520	5 511	5 903	5 900	......	......	5 945	5 955
	......	......	......	......	5 515	......	5 905	5 910	......	......	5 965	5 960
	......	......	......	.....	......	......	5 880	5 856	......	......	5 970	5 990
Means ...	5 216.7	5 218.3	5 300.5	5 299.3	5 518	5 516.8	5 898.5	5 893.5	5 159	5 162.3	5 959.3	5 958.8
Average.	5 217.5		5 299.9		5 517.4		5 896.0		5 160.7		5 959.1	

It is apparent from Tables 37 and 38 that the chemical work of November was not as good as that of October, and, perhaps, one of the most potent reasons therefor is the fact that most of the outdoor work in November was done in bad weather, with the thermometer near zero, Fahrenheit. On the other hand, there is a larger number of titrations in the November work, which would tend to reduce the residual accidental error in the laboratory observations.

As a general rule, two or three chemists performed the titrations in pairs from the same sample, mates appearing on the same horizontal line of the tables.

In closing this section it may be remarked that there is no basis for comparing the gravimetric with the chemical calibration of the calibrator. There is considerable difference between the two, but this is as it should be, as the gravimetric calibration was a measure of the delivery and the chemical calibration was a measure of the total contents of the calibrator. The latter should exceed the former, as it does, but there were no weighings of the total contents of the calibrator, so that there is really no basis for comparison. The evidence is that the two methods can be made to check each other within a few hundredths of 1%, even when the operations are not nearly so refined as the more painstaking work of a laboratory research.

68.—Changes in the Capacity of the Tank.—The general method of calibrating the solution tank was explained in Section 62, *et seq.*

Plate LII is a *fac simile* of the calibration scale taken directly from the gauge-board after it had been removed from the tank at the close of the tests.

The first calibration was made on July 16th, 17th, 18th, 20th, and 21st, 1914. The corresponding scale is indicated on Plate LII by the longest of the graduation marks. The second calibration, July 30th, 31st, and August 1st, 1914, is indicated by the shortest marks; and the third calibration is indicated by the marks of intermediate length.

The first and second calibrations covered the entire scale from graduation 1 to graduation 76, but the third calibration began at graduation 12 and continued to graduation 70, a graduation mark being made at every fifth division.

It will be observed that the first and third calibrations agree remarkably well, and that the second runs with increasing divergence on the side of larger capacity when compared with either of the other two. There is, however, a close agreement between the first and second calibrations from graduation 1 to graduation 45 or 46, and from 52 or 54 to the end of the scale.

Thus all the calibrations agree well with one another except that the first and second calibrations do not agree at points between 45 and 54. Therefore, it must be concluded that there was either an actual change in the calibration scale, from 45 to 54 between the first and second calibrations, followed by a return to original conditions, or an error was made along this part of the scale during the second calibration.

It is not to be imagined that the temperature changes could affect the calibrations so as to cause any such differences. The tank was of 2-in. white pine, with steel hoops. It was about 10 ft. high and about 9 ft. 4 in. in diameter at the top and 9 ft. 8 in. diameter at the bottom, inside measure. The temperatures and corresponding average volumes of the calibrator in the first and second calibrations are given in Table 39. This shows that the changes in the capacity of the calibrator are limited to 0.01 or 0.02 of 1%, which is considerably within the limit of error in the observations during the turbine test. Indeed, the indications are that the volume of the tank was much more consistently constant than could have been anticipated. The third calibration, considered in conjunction with the first, shows that the volume of the tank was the same on both occasions, and the second calibration points to either a change or a slight error between graduations 45 and 54.

TABLE 39.—AVERAGE VOLUME OF CALIBRATOR.

Date.	Graduation.	Temperature, in degrees, centigrade.	Average volume of calibrator, in liters.
FIRST CALIBRATION:			
July 16th	1–14	21.0	229.73
17th	14–32	21.4	
18th	32–45	21.0	
20th	45–60	19.0	239.71
21st	60–76	19.0	
SECOND CALIBRATION:			
July 30th	1–10	19.5	239.715
31st	10–44	19.8	
August 1st	44–76	19.6	
THIRD CALIBRATION:			
October 20th	12–70	13.2	239.665

The writer is inclined to the view that each calibration should be considered as standing by itself, as being the best criterion of the actual volume of the tank at the time of the calibration, any error in utilizing the calibrations being in all probability less than 0.1 of 1 per cent. Therefore, the new calibrations were used as soon as made, the older ones being disregarded in the ensuing turbine tests.

69.—Effect of Painting on the Capacity of the Tank.—Prior to the time of painting the salt-solution tank, it was found that readings, taken night and morning, showed variations of from 0.03 to 0.06 ft. Apparently, there were diurnal changes in capacity due to periodical variations of temperature, humidity, and other atmospheric conditions. Therefore, it was decided to give the tank two coats of white lead.

After the tank had been painted, these fluctuations of water level became markedly lessened, and toward the close of the tests the capacity of the tank became almost fixed.

Table 40, compiled from the record, shows characteristic observations on the level of the water in the tank.

The solution drawn off between the observations of September 25th and 26th was consumed in running some tests on the large centrifugal pump to ascertain the power required for this purpose during a regular turbine test.

TABLE 40.—OBSERVATIONS ON LEVEL OF WATER IN SALT-SOLUTION TANK.

Condition.	Date.	Time.	Gauge reading, in feet.	Remarks.
Before painting.	July 8th.	9.40 A. M.	10.33	
	" 8th.	3 P. M.	10.365	
	" 9th.	3 P. M.	10.38	
After one coat of white lead.	July 15th.	2 P. M.	10.03	
	" 15th.	5 P. M.	10.04	
	" 16th.	8 A. M.	10.02	
After second coat of white lead.	July 20th.	5 P. M.	7.13	
	" 21st.	8 A. M.	7.13	
				Tank filled.
	July 21st.	5 P. M.	10.11	
	" 22d.	8 A. M.	10.10	
				Still later, after refilling.
	July 28th.	4 P. M.	10.53	
	" 29th.	8 A. M.	10.50	
	" 29th.	1.30 P. M.	10.52	
	" 29th.	5.30 P. M.	10.53	
	" 30th.	8 A. M.	10.52	
Another series.	August 5th.	2 P. M.	10.48	
	" 5th.	5.30 P. M.	10.48	
	" 6th.	8.30 A. M.	10.47	
	" 6th.	3.15 P. M.	10.49	
	" 7th.	8 A. M.	10.49	
Last long series on same tank full of salt solution.	September 18th.	5.30 P. M.	9.95	
	" 19th.	8.30 A. M.	9.95	
	" 19th.	12.01 P. M.	9.95	
	" 21st.	8.30 A. M. and P. M.	9.95	
	" 22d	8.30 A. M. and P. M.	9.95	
	" 23d	8.30 A. M. and P. M.	9.95	
	" 24th.	8.30 A. M. and P. M.	9.95	
	" 25th.	8.30 A. M. and P. M.	9.95	
				Some salt solution drawn off.
	September 26th.	8.30 A. M. and P. M.	9.40	
	" 27th.	8.30 A. M. and P. M.	9.40	
	" 28th.	8.30 A. M. and P. M.	9.40	

These observations, together with the results of the calibrations, show that there were no serious variations in the capacity of the salt-solution tank during the series of tests on the turbines.

70.—Care of the Tank.—The principal rule to enforce in the care of the tank is as to the load it carries. If it is always kept filled with salt solution to about the same level during a series of tests, its volume has no tendency to change seriously. If there is no system in this matter, it will be uncertain as to whether there have been serious changes in capacity.

Accordingly, it was a rigid rule, in the tests described, to keep the tank filled with salt solution as continuously as possible. It was always the aim to fill the tank with salt solution immediately after each test,

whenever possible, and on only a few occasions was it allowed to stand over night filled with fresh water instead of salt solution. At no time was it allowed to remain empty, or partly full, during the night, or for any longer time during the day than could be avoided.

The writer attributes the constancy of volume of the salt-solution tank in large measure to the fact that it was kept continuously full of salt solution.

71.—The calibration and accuracy of the electrical instruments will be discussed at the end of Part II.

72.—*Error 9.*—*Time.*—An ordinary watch will keep time sufficiently well for the purpose of turbine testing, even when time is to be determined to within 0.1, or even 0.01 of 1% in precision. There are 1 440 min. in a day, so that 1.44 min., or 1 min., 26.4 sec., represents 0.1 of 1% and 8.64 sec. represents 0.01 of 1 per cent.

Therefore, it may be stated that an ordinary watch in only fair regulation will run within 0.1 of 1%, but it must be in good regulation to run within 0.01 of 1%, and it should be compared with a reliable standard before and after each test, when it is desired to ascertain time more exactly than within 0.1 of 1% in precision.

A much more serious matter than the systematic error in the rate of the watch is the accidental and personal errors of observation. These have been discussed to some extent in Section 63, and will not require extended comment here, except to call attention to the possibilities of watches and chronographs.

There is considerable doubt about the rates of watches operating near large electrical machines, especially in case of direct current. For this reason, it may be advisable, in some cases, to use a chronograph driven electrically by a clock at a distance. A chronograph may be advisable on other grounds, especially where tests can be of short duration, thus saving labor and chemicals where the chemical method is used.

When a watch is used it must be considered that accidental errors approaching 1 sec. are not only possible but probable, even with a stop-watch. Moreover, stop-watches are likely to change their rates by 1 or 2 sec. in 15 or 20 min., unless they are very carefully handled, and there is a high probability of one failing entirely in the course of a test. It is important, therefore, where reliance is placed on stop-watches, to keep several running during a test—not less than three—

where it is important not to lose the observations altogether, and these watches should be rated and compared before and after each use.

In order to secure sufficiently accurate time observations with stop-watches, it must be considered that an accidental error of at least 1 sec. may be made in starting and stopping the watch, or the split-second hand where a split-second watch is used, unless several observers with different watches observe at both start and finish. In this case an idea of the error of observing can be gained from the separate estimates made by the different observers; but, in the case of only one watch, the error must be counted at 1 sec., thus determining the length of a test when the limit of error is decided on.

Therefore, if the limit of the degree of error is 0.1 of 1%, it will be necessary to run the test for 1 000 sec., say, 16 or 17 min. Perhaps 15 min. is the limit, in round numbers. Accordingly, the regular turbine tests described lasted from 15 min. to 1 hour.

The kind of watch most convenient for such work has two hands: one is driven continually from the time the watch is started until it is stopped; the secondary hand moves in coincidence with the primary until it is stopped at any instant by pressure on a small stop. A second pressure on the stop causes the secondary hand to spring into coincidence with the primary again, and these operations may be repeated as often as necessary. Consequently, as many time readings as desired may be made by such a watch, and this without losing account of the correct time, which would be the case by stopping the entire mechanism, as is necessary with many kinds of stop-watches.

Another advantage may be gained, where the time is required in seconds, by having the dial graduated centesimally, each division of what is usually called the minute-scale thus representing 100 sec. Not many watches of this kind have been made, but the writer succeeded in having one divided in this way, and found that it pays well, by reducing the labor of final calculations.

73.—Error 10.—Elimination of the Density of Feed-Water.—The density of the feed-water for the turbine may not be known. If the test is purely for efficiency, where the actual discharge of the turbine is required merely as a factor in the efficiency formula, the density of the feed-water need not be determined, if the units of volume of the calibrator and tank are computed from weights of actual feed-water in conjunction with densities of distilled water.

The error thus made may be eliminated in the efficiency calculation by charging the turbine with theoretical power based on the densities of distilled water and the observed discharge, as given by the observations on the calibration scale at the side of the tank, the counter-errors, thus made, neutralizing each other in the efficiency computation.

Thus, if the density of distilled water is d, that of the actual feed-water is d_1, and the weight of the contents of the calibrator when filled with feed-water is w, the true volume and the computed volume, based on the density of distilled water, are, respectively:

$$\text{True volume of calibrator} = w \div d_1.$$
$$\text{Computed " " } = w \div d.$$

Hence the ratio of the computed to the true volume is $d_1 \div d = 1 + e$, where $100e$ is the percentage of error relative to the true volume. That is, all volumes measured by the calibrator will be affected by the same percentage of error; or, if V is the true discharge of the turbine during a test, then the observed volume will be:

$$\text{Observed volume} = V\ (1 + e).$$

Now, the power with which the turbine will be charged, based on the density of distilled water, and the discharge determined by the calibrator, is:

$$\text{Observed power} = \frac{(V\,[1 + e])\,d\,h}{550} \qquad (141)$$

where h is the head acting on the turbine;

but $$d_1 = d\ (1 + e),$$

as has just been shown, therefore,

$$\left.\begin{array}{r}\text{True power charge-}\\ \text{able to turbine...}\end{array}\right\} = \frac{V\,d_1\,h}{550} = \frac{V\,(1 + e)\,d\,h}{550} \qquad (142)$$

or, this method of calibrating the tank, observing the tests and computing the results, gives the true efficiency, though there may be a small error in the volume of the tank, and therefore of the discharge, as shown by the calibration scale on the gauge-board.

74.—Error 11.—Omission of Weight of Salt.—When the method of weights, Section 10, is used, there is no error due to the omission of the weight of salt, as the total weight of solution passing through the turbine is determined by the equations based on weights (Equations (11) to (28), inclusive). There may be an insignificant error, however, due to

the fact that the tables are based on chemically pure NaCl, whereas the salt used is of commercial grade only. This error is entirely negligible.

In the case of the volumetric method, however, beginning with Equations (38), where the methods of Section 73 are used, the weight of the salt contained in the dosing solution is neglected and results in a corresponding error. This error was negligible in the tests involved herein as the following considerations show:

As a usual case, the discharge of the turbines was about 1 550 cu. ft. per sec., and the rate of dosing was about 0.25 cu. ft. per sec., so that the ratio of dilution was about 1 to 6 200. The strength of the salt solution was about 25%, and its density about 1.18, so that the weight of salt per liter was $0.25 \times 1.18 = 0.295$, say, 0.3 of the weight of distilled water $= 0.3 \times 62.4 = 18.7$, say, 19 lb. per cu. ft. of salt solution, of which one-fourth, or, say, 5 lb., is the rate of salt injection per second. The total weight of water passing through the turbine is approximately

$$1\,550 \times 62.4 = 96\,700, \text{ say, } 100\,000 \text{ lb. per sec.}$$

Therefore, the error introduced by omitting the weight of the salt is, relatively, about $5 \div 100\,000 = 0.00005 = 0.005\%$, which is negligible for work of only 0.1 or even 0.01 of 1% precision.

75.—Error 12.—Sources and Sinks for Chemicals.—In the language of the mathematicians, there may be sources or sinks intermediate to the dosing and sampling stations, where chemical is either introduced into or lost from the water being measured. These gains or losses of chemical may be due to natural causes, such as mineral springs in the bed of the stream, or to artificial causes, such as result from the operations of industries along the channel.

If there is a mineral spring down stream from a leak or sink, where the flow of the spring and the discharge of the leak are about equal, the net result is an introduction of chemical between the two stations. In some such manner there might be a loss of chemical.

Where great accuracy is required, an investigation should be made to ascertain whether there are any such sources or sinks intermediate to the dosing and sampling stations. This is easily accomplished by taking samples, preferably simultaneously at the two stations. If serious discrepancies are found among the titrations of samples, appropriate

alterations in the scheme for measuring the flow of the water must be made.

In some locations it may be found necessary to test samples frequently to see whether there are changes in condition taking place from time to time, and this may have to be done during tests. Of course, it will usually be found possible to avoid this difficulty, but it should be carefully remembered when a discharge measurement is contemplated.

76.—Error 13.—Sources and Sinks for Water.—These are similar to those discussed in the preceding section, but are especially to be feared in the case of a turbine test. It is nearly always the case that there are leaks of more or less consequence through the concrete walls of penstocks and turbine pits. Besides this, in many cases the draft-tubes fail to discharge water throughout the entire cross-section. This results in eddies and negative currents which may introduce water containing more or less salt than should result from the salt injection intended for the test.

One of the easiest ways to examine this source of error is to shovel a considerable quantity of salt into the tail-race down stream from the tail-race sampling pipes. During the time that the salt is being shoveled into the tail-race the sampling pumps are kept in operation and samples of the tail-water are secured in the tail-race while samples of normal head-water are taken simultaneously in the head-race. If all samples titrate alike, it is safe to say that no tail-water was washed back into the tail-race during the test. A single test of this kind, however, may not be conclusive for all gate openings and turbine speeds. It may be necessary to repeat the test under a variety of conditions.

The best safeguard against this difficulty is to place the tail-sample pipes at a point in the system where the water is known to flow in only one direction throughout the entire cross-section. These pipes may even be placed in the head-race a short distance below the dosing pipes, but in this case, as the advantage of the mixing power of the turbine itself is lost, it will be necessary to design and operate the dosing system very carefully so as to secure as perfect a mixture as possible at the outset. See Section 56.

This method has been used with a fair degree of success in a case where the dosing solution was supplied to the distributing and sprinkling system by a centrifugal pump of insufficient capacity.

It is quite important in such cases to have a centrifugal pump and sprinkling system which will force the jets of water to a considerable distance laterally, in order to secure good mixtures.

77.—Error 14.—Errors in Tables and Data.—There may be some slight errors in the salt tables and diagrams (Table 1 and Plates XLVIII, XLIX, and L). These may be relatively large for the smaller values of the quantities tabulated. Thus, the theorem of Section 23, that a molecule of salt always displaces a portion of the water in which it is dissolved may need revision when more accurate data are available.

However, the results of the tests and tank calibrations described herein are of such a character that it may be concluded there was no error in respect to the accuracy of the tables which affects the results appreciably.

78.—Error 15.—Theory.—Simplified Calculations.—Advantages of Measuring Samples by Weight.—One of the best chances for error lies in the calculation itself. The simpler the calculations are made the less chance there is for errors. In Section 10 attention was directed to the system of weighing samples rather than measuring them by volume. In order to make the advantages of this method more apparent, the following calculation of the weight of the contents of the calibrator (November 25th, Table 35) is given, with a final reduction to volume for the purpose of checking the result determined in that table, 241.02 liters at 4° cent. The check is not absolutely perfect, owing principally to the fact that in computing the data for the calculation corresponding to that of the table, the weight, W'_2, has been taken at 860 grammes and the weight, W', at 10 grammes, instead of 1 003.6 and 11.671 grammes, respectively, which correspond more exactly to the figures of the table. Had these figures been taken, the ratio, F', would have been 85.99, instead of 86, and the result would have checked the figures of Table 35 very closely. The rounded weights have been chosen, as it is quite probable that the samples would be weighed in about that manner, it being a simple matter to adjust the quantity of solution to a given weight. It would be better practice, however, to adjust the weights so that all samples would titrate alike. Hence the titration, t, has been inserted, which represents the titration of 11.7 grammes of salt solution, this number being, aproximately, a multiple by ten of the density. The advantages of this method will undoubtedly

be appreciated after an examination of Table 41, which, with but few additional reductions, would include all the necessary data and the final calculations, thus displacing both Tables 34 and 35. These latter tables could be made somewhat simpler than they are, but, in the volumetric method, it is better to reduce all values to the temperature of the feed-water (or tail-water, which is at the same temperature as the head-water, within the limits of ordinary accuracy), so as to make the calculations consistent in appearance as well as in fact.

TABLE 41.—Illustrating the Advantages of the Method of Weighing Samples.

(The data have been computed from those of Tables 34 and 35 so as to represent closely the necessary data and calculations by the method of weights, Section 10, as applied to the calibration of the iron pipette, or calibrator, November 25th, 1914. All items marked D in the last column are parts of the data; all others result from computation. It will be seen that the temperature does not enter until it becomes necessary to pass to volumes.)

Explanation.	Line No.	Symbol.		Data.
Weight of salt solution taken for dilution with distilled water at temperature of laboratory, which temperature is not required	1	....	11.7 grammes.	D
This salt solution is diluted with distilled water until the weight of the solution is	2	....	1 000 grammes.	D
Weight of dilution taken for titration	3	....	10 grammes.	D
Mean titration of above samples	4	t	53.212 c. c.	D
Mean titration per kilogramme = 100 000 × Line 4 ÷ Line 1	5	v	454 800 c. c.	
Mean titration per 10-gramme sample of mixture from tank calibrator	6	t_2	53.273 c. c.	D
Mean titration per kilogramme of tail-water	7	v_2	5 327.3 c. c.	
Mean titration per 10-gramme sample of special dilution	8	t_2	53.168 c. c.	D
Mean titration per kilogramme of special dilution	9	v'_2	5 316.8 c. c.	
Line 7 — Line 9	10	$v_2 - v'_2$	+ 10.5 c. c.	
Line 5 — Line 9	11	$v - v'_2$	449 491 c. c.	
Line 10 ÷ Line 11	12	y	0.0000234	
Weight of normal head water for special dilutions	13	W'_1	850 grammes.	D
Weight of salt solution for special dilutions	14	w'	10 grammes.	D
Ratio of dilution for special dilutions $[(W'_1 + w') \div w']$	15	F'	86.0	
Value of $(F' - 1)\, y = 85\, y$	16	$(F' - 1)\, y$	0.00199	
Value of the ratio of dilution in calibrator (by weights) found by dividing value of Line 15 by $1 + 0.00199 = F$	17	F	85.829	
Weight of salt solution introduced into calibrator, which was then nearly full of normal canal water, and immediately afterward filled completely with normal canal water, the whole being agitated until the mixture was uniform	18	w	2.8137 kg.	D
W_2 = total weight of mixture in calibrator	19	$w\, F$	241.50	
Density of mixture at 4° cent	20	d_2	1.0019	D
Volume at 4° cent. = $W_2 \div d_2$	21	Q_2	241.04	

It will be at once apparent that if it was the weight of the solution in the calibrator which was required, the calculations of Table 41 would be entirely independent of temperature observations, and therefore independent of temperature corrections. In the case of turbine tests, the only observation requiring a volumetric observation would be the determination of the weight of salt solution introduced into the head-water. This would probably best be done by observing the volume of salt solution consumed, exactly as was done during the tests described, with accurate weighings of standard volumes taken as samples from the sampling cocks in the 3-in. salt-solution supply pipe. These should be quickly measured and weighed on the spot and then poured back into the salt-solution tank. Several of these weighings should be made during the test, and the results should check well within the limit of permissible error. The mean of the results divided by the standard volume will be the density by which the total volume, or volume per second, is to be multiplied in order to get the total weight of solution consumed, or the weight per second, as the case may be.

The calculations of Table 41 are based on the last of Equations (27), Section 13, and the data have been reduced from those of Tables 34 and 35 by using Equations (38), Section 18.

79.—Error 16.—Determination of Acting Head.—It is the opinion of the writer that the determinations of acting head were the most doubtful of the data taken during the tests. The elevations of head- and tail-water could be determined quite accurately according to the requirements of the contract, but the real question at issue is as to the constant error involved in this method; and this leads to the more general question: How should head be measured for the purpose of turbine tests?

If one wishes to determine the energy actually entering the waterway for the turbine to be tested, it will be necessary to determine the actual kinetic energy of the water and the actual mean elevation of the water surface. If one wishes to determine the actual hydraulic efficiency of the installation—charging the wheel with the energy entering the waterway and deducting the energy which leaves the draft-tube—similar quantities for the tail-water must be determined. It would appear, however, that the latter operation should not be im-

posed by the terms of efficiency contracts, as it is urgently demanded that as little loss as possible result from tail-race flow.

Probably the contract should charge the turbine with all the energy flowing into the head-water passages and credit it with only such energy as it delivers at its shaft coupling. Contract efficiency and hydraulic efficiency, in the sense used here, are two entirely different things.

Now, it is highly probable that a sufficiently close estimate of the kinetic energy of the entering water can be computed from our practical knowledge of the character of flow of water in head-races, thereby avoiding the necessity for intricate current-meter or Pitot-tube surveys of water flow in such water passages.

Take, for example, the case of a head-race of width, $W = 2R$, when the water is flowing with parabolic distribution of velocities, looked on in plan, the axis of the parabola coinciding with the axis of the race. The total kinetic energy in the race would then be

$$E = \tfrac{1}{2}\int v^2\, d\, m = \frac{\gamma\, b}{2\, g}\int_{-R}^{+R}\left(V - \frac{y^2}{4\, a}\right)^3 d\, y = \frac{16}{35}\,\frac{\gamma\, b\, R\, V^3}{g}. \quad (143)$$

where $y^2 = 4\, ax$, $R^2 = 4\, aV$, $v = V - x$, $dm = \dfrac{\gamma\, v\, d\, A}{g}$, and $dA = b\, dy$, according to usual notation, $b\, W = A$ being the area of the cross-section of the race, b being the depth of water in the race, γ = specific weight of water, g = acceleration of gravity, and V being the maximum velocity in the section along its axis.

By a well-known property of the parabola, the mean velocity, V_m, in this problem, must be two-thirds of the maximum velocity, that is, $\frac{2}{3}\, V = V_m$. Hence, the discharge, Q, is equal to $A\, V_m$, the weight of the discharge is $\gamma\, A\, V_m$, and the total energy is

$$E = \frac{54}{35}\,\frac{\gamma\, b\, R\, V^3{}_m}{g} = \frac{27}{35}\,\frac{\gamma\, A\, V^3{}_m}{g}.$$

Consequently, as the mean velocity head is equal to the total energy divided by the weight of the discharge, we must have

$$\left.\begin{array}{l}\text{Mean velocity head for}\\ \quad\text{parabolic distribu-}\\ \quad\text{tion in horizontal}\\ \quad\text{planes}\end{array}\right\} = \frac{54}{35}\,\frac{V^2{}_m}{2\, g} = 1.54\,\frac{V^2{}_m}{2\, g} \ldots\ldots (144)$$

It is quite apparent, therefore, that for anything like ordinary distribution of velocities, the mean velocity head is greater than the head due to the mean velocity in the raceway.

If we take the case of paraboloidal distribution in a circular race, it may be shown that

$$\left.\begin{array}{l}\text{Mean velocity head for paraboloidal}\\ \quad\text{distribution in circular races run-}\\ \quad\text{ning full}\end{array}\right\} = 2\,\frac{V^2_m}{2\,g}\,\ldots\ldots(145)$$

The case of triangular distribution in horizontal planes in a rectangular race is identical with paraboloidal distribution in circular races running full, in so far as the relation between mean velocity head and head of mean velocity is concerned, that is, in either case the mean velocity head is twice the head due to the mean velocity.

At first sight, these last results do not seem to agree with parabolic distribution, in that the latter would appear to have relatively more high velocities than triangular distribution, and the numerator of mean velocity head is made up of cubes of the elementary velocities while the denominator contains first powers only. Apparently, the coefficient should be higher for parabolic than for paraboloidal or triangular distribution. The deception lies in the fact that V_m is greater relatively to V in parabolic distribution than it is in the other cases. Thus the actual mean velocity heads for a given maximum velocity, V, in the two cases are:

$$\left.\begin{array}{ll}\text{Parabolic distribution} & = 0.686\,\dfrac{V^2}{2\,g}\\[2ex] \text{Paraboloidal or triangular distribution} & = 0.50\,\dfrac{V^2}{2\,g}\end{array}\right\}\ldots(146)$$

Thus, for a given maximum velocity, the mean velocity head is greater for parabolic than for paraboloidal or triangular flow, and the converse is true for a given mean velocity.

80.—Actual Distribution of Velocities in Raceways.—The foregoing assumptions are not sufficiently exact to represent accurately the actual conditions of flow in raceways. It is much more likely that the flow would approximate the conditions which may be indicated by adding a constant velocity, V_1, to the velocities of parabolic distribution in horizontal planes. Thus, if $S = V + V_1$ be substituted for V in the foregoing integral, Equation (143), V_1 being constant,

$$E = \frac{1}{2}\int v^2\,d\,m = 2\,\frac{\gamma\,b\,\sqrt{a}}{g}\int_0^V (S - x)^3\,d\,(x^{\frac{1}{2}})$$

therefore,

$$E = \frac{2\,\gamma\, b\,\sqrt{a}}{g}\left(k^3 - k^2 + \frac{3}{5}k - \frac{1}{7}\right) V^{\frac{1}{2}} \ldots\ldots(147)$$

where $k = S \div V = (V + V_1) \div V$ and $y^2 = 4\,ax$.

By noting that $2\sqrt{a}\,V^{\frac{1}{2}} = R$, and $2\,R = W$, Equation (147) may be written

$$E = \frac{\gamma\, b\, W\, V^3}{2\,g}\left(k^3 - k^2 + \frac{3}{5}\,k - \frac{1}{7}\right) \ldots\ldots\ldots(148)$$

Moreover, if V_m is the mean velocity in the raceway,

$$V_m = \frac{2}{3}V + V_1 = S - \frac{V}{3} = \left(k - \frac{1}{3}\right) V \ldots\ldots(149)$$

and the discharge is

$$Q = b\,W\,V_m = \left(k - \frac{1}{3}\right) b\,W\,V \ldots\ldots\ldots(150)$$

from which the mean velocity head is evidently

$$\frac{E}{Q\,\gamma} = \frac{\left(k^3 - k^2 - \frac{3}{5}k - \frac{1}{7}\right)}{\left(k - \frac{1}{3}\right)} \frac{V^2}{2\,g} \ldots\ldots\ldots\ldots(151)$$

But, from Equation (149),

$$V^2 = \frac{V^2{}_m}{\left(k - \frac{1}{3}\right)^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots(152)$$

so that, in terms of V_m, mean velocity head

$$= \frac{k^3 - k^2 + \frac{3}{5}\,k - \frac{1}{7}}{\left(k - \frac{1}{3}\right)^3} \frac{V^2{}_m}{2\,g} \ldots\ldots\ldots\ldots\ldots(153)$$

This equation will approximate more or less closely the actual mean velocity head when the value of k approximates the actual value, based on the observed ratio of S to V.

Take, for example, the actual distribution in the tail-races observed by the writer in 1911.* In the first of these two tables, $S \div V = \frac{306}{306 - 150} = 1.96$. In the second, $S \div V = \frac{311}{145} = 2.14$,

* *Transactions*, Am. Soc. C. E., Vol. LXXVI, pp. 827-828, Tables 3 and 4.

the mean being 2.05, or approximately 2. Again, in Test *G*, of the present turbine tests, the corrected current-meter observations furnish the distribution shown in Table 42.

TABLE 42.—VELOCITIES AT ONE HUNDRED UNIFORMLY DISTRIBUTED METER POINTS IN HEAD-RACE OF UNIT NO. 13, TEST *G*.

(The unit is $\frac{1}{100}$ ft. per sec., except in the final averages, where it is 1 ft. per sec.)

Depths.	Velocities.									
1	260	261	286	256	261	259	257	286	317	351
2	301	294	292	265	263	253	267	268	355	323
3	366	300	300	287	278	258	268	259	294	272
4	304	293	302	261	254	266	254	268	300	312
5	360	318	297	270	249	260	258	279	297	308
6	338	298	263	256	269	253	275	262	296	302
7	325	271	256	259	276	238	278	308	267	304
8	265	215	256	250	234	238	291	295	267	248
9	244	261	266	269	227	230	270	307	263	199
10	156	218	211	208	211	225	248	310	350	000
	2 919	2 729	2 729	2 581	2 522	2 480	2 666	2 842	3 006	2 619

Total................270.93

V_m = mean velocity = 2.7093.
V^3_m = 19.8871, say, 19.89.

In this distribution there are two extremely low velocities which probably should be neglected. Perhaps a velocity of 2 would be a fair minimum, so that $k = 366 \div (3.66 - 2) = 2.2$. It might be assumed, therefore, that:

In tail-races of 1911 tests, $k = 2.0$;
In head-races of 1914 tests, $k = 2.2$;

the corresponding values of the mean velocity head, by Equation (153), being:

$$\left.\begin{array}{ll} \left.\begin{array}{r}\text{Mean velocity head in tail-}\\ \text{race, tests of 1911......}\end{array}\right\} & = \dfrac{4\,779}{4\,375}\,\dfrac{V^2_m}{2\,g} = 1.09\,\dfrac{V^2_m}{2\,g} \\ \left.\begin{array}{r}\text{Mean velocity head in head-}\\ \text{race, tests of 1914......}\end{array}\right\} & = \dfrac{699}{655}\,\dfrac{V^2_m}{2\,g} = 1.07\,\dfrac{V^2_m}{2\,g}\end{array}\right\} \ldots (154)$$

Consequently, it may be said that the mean velocity head in the tail-races, down stream from the stilling racks, in the tests of 1911,

on a horizontal unit driven by a string of six turbines, was nearly the same relative to the head due to mean velocity as in the head-races, up stream from the trash racks, in the tests of 1914, on a vertical double-runner unit discharging nearly as much water as the turbine unit on the former occasion.

In the case where the maximum is about twice the minimum velocity in a raceway, it may be said, therefore, that, as an average,

$$\left.\begin{array}{r}\text{Mean velocity head, where } k = 2 \text{ and} \\ V_m = \text{mean velocity in raceway...}\end{array}\right\} = 1.09 \frac{V^2_m}{2g} \ldots\ldots (155)$$

81.—Mean Velocity Head During the Tests.—We are now in a position to compute the actual mean velocity head in the head-races during any of the tests. The elevation of the floor of the head-race was 176 ft. above sea level. In Test *G* the water surface was at Elevation 199, the depth, therefore, being 23 ft. The width of the race is about 27 ft., so that the area of section was about 621 sq. ft. The discharge for this test was about 1 662 cu. ft. per sec., which corresponds to a mean velocity of 1 662 ÷ 621 = 2.68 ft. per sec. The head due to this velocity is 0.112 ft., which makes the mean velocity head = 0.12 ft.

Test 41 was taken very near the point of maximum efficiency. The discharge was 1 520 cu. ft. per sec., the elevation of the head-water was 196.7, the depth, consequently, was 20.7 and the area 560 sq. ft. Thus, the head due to the mean velocity was 0.115 ft., and the mean velocity head, by Equation (155), was 0.125 ft.

In some of the tests the elevation of the head-water was observed at the crown of the arch, at the point where the water changes its flow to descend to the wheel. At this point the water piled up, by reason of the change in direction, the surface water thereby coming very nearly to rest. Theoretically, this should equal the velocity head of the water before it is retarded. Table 43 gives the results of such observations as were made during the tests; and it will be seen that the rise of water due to its stoppage approximates closely the rise computed by the equations of the foregoing theoretical and practical study deduced in another way.

82.—Actual Mean Velocity Head.—Regardless of all theory, one may observe the individual velocities at the various points in a cross-section, and compute the velocity head at each point, thus enabling a

determination of the mean velocity head from direct calculations. This procedure would be indicated by the following equation, which is self-explanatory:

$$\left.\begin{array}{r}\text{Mean velocity head in}\\ \text{raceway, where } V_m\\ = \text{the mean velocity}\end{array}\right\} = \left.\begin{array}{l} \dfrac{E}{\gamma\, Q} = \dfrac{\dfrac{\gamma}{2\,g}\,\Sigma\, v^2\, d\, Q}{\gamma\, Q} \\ = \dfrac{1}{2\,g}\,\dfrac{\Sigma\, v^3\, d\, A}{A\, V_m} \\ = \dfrac{\dfrac{d\,A}{A}\,\Sigma\, v^3}{V^3_m}\;\dfrac{V^2_m}{2\,g} \end{array}\right\} \ldots (156)$$

In words, the theorem runs:

The ratio of the mean velocity head of flowing water to the head due to its mean velocity is equal to the ratio of the mean of the cubes of the velocities in the cross-section to the cube of the mean velocity.

Therefore,

$$\frac{k^3 - k^2 + \frac{3}{5}\,k - \frac{1}{7}}{\left(k - \frac{1}{3}\right)^3} = \frac{\dfrac{d\,A}{A}\,\Sigma\, v^3}{V^3_m} \;\ldots\ldots\ldots\ldots.(157)$$

TABLE 43.—DIFFERENCES IN ELEVATION BETWEEN MOVING AND STILL WATER IN HEAD-RACES OBSERVED DURING CERTAIN OF THE TESTS ON UNITS NOS. 11 AND 12.

(The unit is 1 ft.)

UNIT NO. 11.				UNIT NO. 12.			
Test.	Average elevation of head-water by slanting rod.	Average elevation of head-water by rods and boards.	Difference in elevation.	Test.	Average elevation of head-water by slanting rod.	Average elevation of head-water by rods and boards.	Difference in elevation.
111	195.46	195.29	0.17	200	195.93	195.68	0.25
112	194.80	194.63	0.17	201	196.12	195.92	0.20
113	195.84	195.67	0.17	202	195.50	195 33	0.17
114	195.26	195.09	0.17	203	195.72	195.51	0.21
115	194.88	194.72	0.16	204	195.98	195.80	0.18
116	195.98	195.79	0.19	205	195.31	195.19	0.12
117	196.87	196.67	0.20	206	195.10	194.98	0.12
118	197.51?	197.19	0.32?	207	195.25	195.12	0.13
119	195.11	194.94	0.17	208	195.33	195.18	0.15
120	196.04	195.83	0.21	209	195.75	195.66	0.09
121	195.12	195.00	0.12	210	195.97	195.82	0.15
122	195.78	195.67	0.11	211	196.52	196.34	0.18
				212	194.94	194.86	0.08

when the vertical projection of the velocities on a horizontal plane may be represented by a rectangle capped by a segment of a parabola, the velocities themselves being supposed to be horizontal and equal in any vertical, and k being the ratio of the combined height of the rectangle and segment (maximum velocity) to the height of the segment.

Applying Equation (156) to the cases of the tests of 1911 and 1914, by computing the mean of the cubes of the velocities in the tables and dividing by the cube of the mean velocities, the following may be determined as the actual coefficients for the head due to mean velocity:

$$\left.\begin{array}{l} \text{Tests of 1911, coefficient} = \dfrac{14.5}{13.5} = 1.07 \\[2ex] \text{" " 1914, "} \quad = \dfrac{21.30}{19.89} = 1.07 \end{array}\right\} \ldots\ldots (158)$$

The values agree very closely with those deduced above, based on less direct assumptions. The general conclusion is that Equation (157) gives the value of the coefficient in terms of k with a considerable degree of precision, where the flow is not unusual in character.

83.—Personal Errors and Carelessness.—Shaking Flasks.—It requires experience for a person to develop what is technically called a personal error. Such an error is a systematic error in that it always occurs in the same sense and in about the same degree under given circumstances. In many cases this error may be eliminated by properly arranging the methods of observing. For example, if a chemist has acquired by experience the fixed habit of reading a burette in such a manner that all observations are too high by 0.02 c.c., then the error is eliminated when he subtracts the lower from the higher reading to determine the number of cubic centimeters of silver nitrate consumed in a titration. Hence, if it requires two gauge readers to determine the acting head during a turbine test, it would be advisable to have them change stations during the test, so as to have equal numbers of head and tail-gauge readings by each observer. Again, in the case of pipettings, two chemists may handle their instruments in different ways, so that pipettings by one of them will always be slightly larger than by the other. Such an error may be eliminated from discharge measurements by the method of group titrations, discussed in Section 48.

Carelessness, however, can be guarded against only by its prevention. Ignorance is closely akin to carelessness, and must be prevented

by having every observer understand exactly what he is to do and how it is to be done. The rules must be specific, pertinent, and concise. Therefore, the person who undertakes to conduct a chemical test must have every detail of it at his command. He must go over the ground thoroughly in advance, and provide for every contingency.

Nothing illustrates the importance of these remarks better than the difficulties encountered in securing accurate chemical work during the first few of the turbine tests described herein. It was apparent that something was radically wrong. After the first week's work, the writer devoted an entire day in the laboratory to repeating some work which was manifestly in error, the result being the discovery that the dilutions were not being thoroughly mixed by sufficient agitation of the flasks containing them.

All chemists know that flasks must be shaken in order to secure perfect mixtures, but it does not seem that there is any definite rule as to how many times, or how vigorously, the shaking must be administered. It would seem that this should depend on the shape and size of the receptacle in which the mixture is to be effected and the manner of shaking.

In the case of an ordinary stoppered liter flask with a long neck, about 0.6 in. in diameter, it requires from 25 to 30 double shakes or inversions, with time for the air to rise to the surface of the liquid in each position, in order to secure the uniform mixture of 10 c.c. of nearly saturated salt solution with about 990 c.c. of distilled water. In order to insure results, the rule was adopted that such operations should require 40 double inversions, with sufficient time allowance in each position for the air to rise to the surface. In the case of a different size or shape, the rule will probably require modification.

The calibrations of the flasks are given in diagrammatic form in Fig. 6.

84.—Effect of Variations in Discharge on the Concentration of Tail-Water Samples.—The discharge of a turbine is likely to vary, even when efforts are made to maintain uniform conditions during a test. It is necessary, therefore, to show that the method of testing and sampling does not introduce an error.

First, it is necessary to show that, when a tail-race sample is pumped into a sample bottle at a uniform rate, the resulting concentration is the time-average concentration of the water at the sampling pipe, in-

stead of the mean concentration relative to all the volume of the water passing the pipe.

The time-average concentration is evidently

$$\int \frac{c_2\, d\, t}{T} \qquad (159)$$

and the total quantity of salt in the sample is

$$\int c_2\, x\, d\, t \qquad (160)$$

where x is the rate at which the sample is pumped into the sample bottle, which rate is supposed to be constant. The total volume of the sample is

$$\int x\, d\, t \qquad (161)$$

CURVES SHOWING VOLUMES OF FLASKS AT VARIOUS TEMPERATURES,
As Derived from Direct Volumetric Comparisons with the Contents of Flask No. 1456 or Flask No. 11, which were Calibrated by the U.S. Bureau of Standards.
Note:-Flask No. 1456 was calibrated by Arthur Schweier at the Chemical Laboratory of the University of Pittsburgh, and its Contents at 19.7°C. were found to be 1002.68 c.c., as compared with 1003.00 c.c. given by the U.S.B.S.

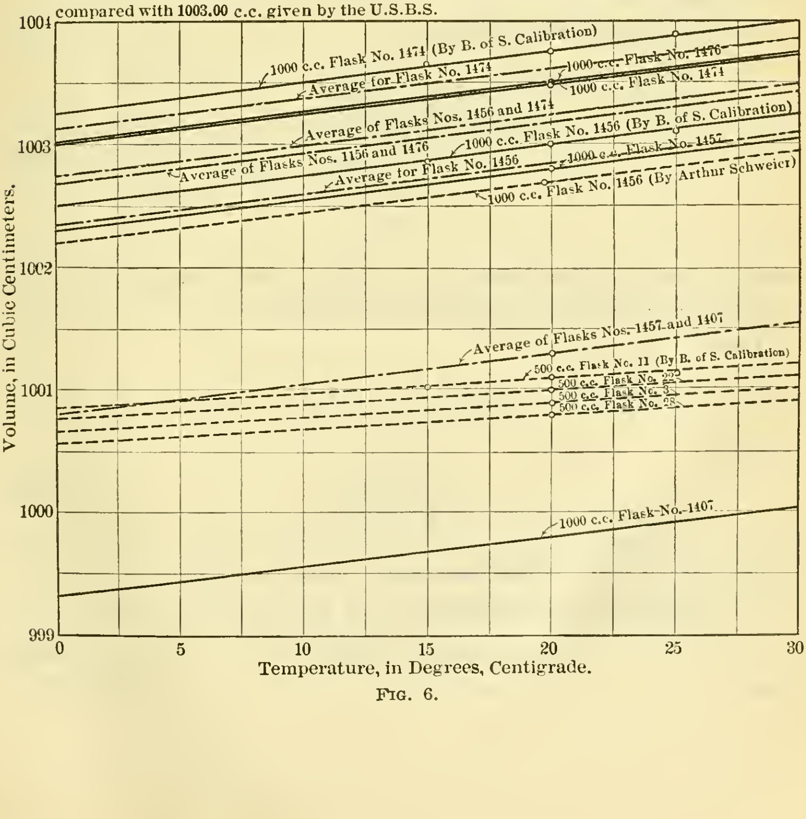


FIG. 6.

Therefore, the concentration of the sample is

$$c_2 = \frac{\int c_2\, x\, d\, t}{\int x\, d\, t} = \frac{\int c_2\, d\, t}{T} \quad \text{.................(162)}$$

as x is constant.

By Equation (159), however, this is a time average. Hence, the proposition is proved that the concentrations of samples pumped at uniform rates are time averages.

We may now investigate the relations existing between time averages and volume averages, with especial reference to tail-water concentrations.

Let c_{2m} be the average concentration with reference to the total volume of discharge during a given time, T, which is supposed to be an interval during which the discharge, Q_2, varies, and which is included as part of the time of a test. Let c_{2t} be the time average of the concentration during the same period. The object is to find the relation of c_{2m} to c_{2t}.

Then, in mathematical language,

$$\left.\begin{aligned} c_{2\,m} &= \frac{\int c_2\, d\, D}{\int d\, D} \\ \text{and} \qquad c_{2t} &= \frac{\int c_2\, d\, t}{T} \end{aligned}\right\} \quad \text{.....................(163)}$$

where D is the total discharge at any given instant during the interval, T, reckoned from the beginning of the interval.

Thus

$$Q_2 = \frac{d\, D}{d\, t},$$

whence

$$c_{2\,m} = \frac{\int Q_2\, c_2\, d\, t}{\int Q_2\, d\, t} \quad \text{..................... (164)}$$

Q_2 being the tail-race discharge at any instant, as usual.

But, by Equation (99), as $R = Q_2 \div q$,

$$Q_2\, c_2 = m\, q + Q_2\, c_1 \quad \text{....................(165)}$$

where

$$m = c - k\, c_1.$$

Therefore, by Equations (162), (163), and (164),

$$\left.\begin{aligned} c_{2m} &= \frac{\int (m\,q + Q_2\,c_1)\,d\,t}{\int Q_2\,d\,t} = \frac{m\,q\,T}{\int Q_2\,d\,t} + c_1 \\ c_{2t} &= \frac{\int \frac{m\,q + Q_2\,c_1}{Q_2}\,d\,t}{T} = \frac{m\,q \int \frac{d\,t}{Q_2}}{T} + c_1 \end{aligned}\right\} \ldots\ldots (166)$$

From the nature of the integrals, it is apparent that the discrepancy between c_{2m} and c_{2t} depends on the manner in which the discharge varies during the interval over which the averages are to be computed. Suppose, for example, that

$$Q_2 = Q_0 + a\,t \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (167)$$

where a is constant and Q_0 is the discharge at the beginning of the interval, t being the time reckoned from that instant.

Then we may show that

$$c_{2m} = \frac{m\,q}{Q_0 + a\,\frac{T}{2}} + c_1 = \frac{m\,q}{Q_0}\;\frac{1}{1 + \frac{1}{2}\,z} + c_1 \ldots\ldots\ldots (168)$$

and

$$c_{2t} = \frac{m\,q}{a\,T} \log.\left(1 + \frac{\Delta\,Q}{Q_0}\right) = \frac{m\,q}{Q_0}\,\frac{1}{z} \log.\,(1 + z) + c_1$$

where $\Delta\,Q$ is the total change in discharge during the interval in question and z is the relative change, $\Delta\,Q \div Q_0$.

But, by Equation (165), we have

$$\frac{m\,q}{Q_0} = c_0 - c_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (169)$$

where c_0 is the value of c_2 at the beginning of the interval, whence,

$$\left.\begin{aligned} c_{2m} &= \frac{c_0 - c_1}{1 + \frac{1}{2}\,z} + c_1 \\ c_{2t} &= \frac{c_0 - c_1}{z} \log.\,(1 + z) + c_1 \end{aligned}\right\} \ldots\ldots\ldots\ldots (170)$$

Now it is apparent that these results are independent of the length of the interval, the only restriction being that the change in discharge shall be at a uniform time rate, though this rate may be relatively fast or slow. The values of the averages depend otherwise only on the relative change in discharge which takes place during the time considered.

Let it be supposed, therefore, that the relative changes of discharge are, respectively, 0.1, 0.2, 0.3, 0.4, and 0.5, first in the sense of increase and then the reverse. The resulting comparisons of the values of v_{2m} and v_{2t} are given in Table 44, together with the calculations.

TABLE 44.—CALCULATION OF THE RATIOS OF THE TIME AVERAGE CONCENTRATIONS IN THE TAIL-RACE TO THE CORRESPONDING MEAN CONCENTRATIONS FOR ANY INTERVAL OF TIME DURING WHICH THE DISCHARGE INCREASES OR DECREASES AT A UNIFORM RATE EXPRESSED AS A FUNCTION OF THE RELATIVE CHANGE IN DISCHARGE DURING THE INTERVAL. THE CONCENTRATION OF THE TAIL-WATER AT THE BEGINNING OF THE INTERVAL IS ASSUMED TO BE THE EQUIVALENT OF 110 C.C. OF SILVER NITRATE TITRATION PER LITER, AND THE NORMAL HEAD-WATER CONCENTRATION 25 C.C.

(Thus $v_0 - v_1 = 85$ c.c.)

z	$\frac{1}{z}$ log. $(1+z)$	$\frac{1}{1+\frac{1}{2}z}$	v_{2t} c.c.	v_{2m} c.c.	$v_{2t} - v_{2m}$ c.c.	$\frac{v_{2t}}{v_{2m}}$	Percentage of error.
0.5	0.8109	0.8000	93.93	93.00	0.93	1.0100	1.0
0.4	0.8412	0.8333	96.50	95.83	0.67	1.0070	0.70
0.3	0.8745	0.8696	99.33	98.92	0.41	1.0041	0.41
0.2	0.9116	0.9091	102.49	102.27	0.22	1.0022	0.22
0.1	0.9531	0.9524	106.01	105.95	0.06	1.0006	0.06
0.05	0.9758	0.9756	107.94	107.93	0.01	1.0001	0.01
0.00	1.0000	1.0000	110.00	110.00	0.00	1.0000	0.00
— 0.05	1.0258	1.0256	112.19	112.18	0.01	1.0001	0.01
— 0.1	1.0540	1.0526	114.59	114.47	0.12	1.0008	0.08
— 0.2	1.1155	1.1111	119.82	119.44	0.38	1.0032	0.32
— 0.3	1.1890	1.1765	126.06	125.00	1.06	1.0085	0.85
— 0.4	1.2770	1.2500	133.55	131.25	2.30	1.0175	1.75
— 0.5	1.3862	1.3333	142.83	138.33	4.50	1.0325	3.25

It is very evident from the array of figures in Table 44 that any change in discharge during a test causes an over-estimate of concentration of tail-water where the samples are taken at a uniform rate, but that the error introduced is less than 0.1% for variations as great as 10% of the discharge, and less than 0.01% for variations less than 5% of the discharge. Moreover, these errors will be greatly diminished when the power is constant during a part of the test, so that they are entirely negligible in the tests now being examined, as the maximum variation of power during any of the decisive tests was less than 5 per cent.

The general truth, that time averages of concentrations for variable discharges with uniform dosing are too large as compared to volume averages, may be seen without recourse to computations.

First, suppose the discharge to increase during a certain interval of time. Then the concentration must decrease during that interval. Now, a time average gives equal weights to the momentary concentrations at all instants, and a volume average, in this case, gives greater weight to the momentary concentrations as time progresses. Therefore, for increasing discharge, the volume average gives greater weights to the weaker concentrations and results in a smaller value than does a time average.

Second, suppose the discharge to decrease during a certain interval of time. Then the concentration must increase during that interval; but the time average gives equal weights to the momentary concentrations at all instants and a volume average, in this case, gives less weight to the momentary concentrations as time progresses. Therefore, for decreasing discharge, the volume average gives smaller weights to the stronger concentrations, and results in a smaller value than does a time average.

Therefore, for variable discharges, with uniform dosing, the time average is always greater than the volume-average concentration.

This truth may be seen very clearly in still another manner. Suppose two tail-water samples are being accumulated in two sample bottles, the first being pumped into its bottle at a uniform rate while the second is pumped into its bottle at a rate proportional to the discharge, both samples being started at the same rate.

Then, if the discharge is increasing, it is clear that the first bottle is not receiving enough of the weaker solutions as time progresses; but, if the discharge is decreasing, it is clear that the first bottle is receiving too much of the stronger solutions as time progresses.

It follows, in either case, that the concentration is greater in the first than in the second sample bottle.

85.—Securing Volume Average Concentrations.—In determining on the method of sampling with small pumps drawing their supply from suctions in the tail-race, it was realized that time averages of concentration would result, rather than volume averages; but it was thought that it would be better to have something which was definite than to try to secure a volume average in the absence of any method for pumping, or otherwise taking, samples from the tail-race at rates which varied in direct proportion to the velocity of the tail-water at the inlet of suctions.

In considering the various methods of taking continuous samples, both the vacuum and gravity methods were discussed. The power-house was equipped with a vacuum system, the vacuum being derived from one or more of the draft-tubes. The samples might have been sucked up by this vacuum and tapped off at various points by a simple device. An inverted caisson might have been constructed and pumped out, or the sump well in the power-house might have been used, in either of which a system of pipes discharging the samples continuously by gravity into the sample bottles could have been installed.

It was thought, however, that the small pumps would be more reliable as to the constancy of rate than either of these methods, and they were therefore adopted.

If it is desired to secure volume-average samples, both as to time and place, in the tail-race, the object may be accomplished by installing the suctions of the pumps in the form of Pitot tubes with static orifices attached, the Pitot tubes pointing up stream with their axes parallel to the center line of the race.

Both the Pitot tubes and static orifices must be connected by small tubes leading from holes in their walls to the static and dynamic gauges arranged alongside the corresponding pumps, or other devices for drawing out the samples.

An attendant regulates the rate of sampling at each pump or sampling device, so that the dynamic and static columns, which are drawn up in the usual manner on the graduated scales corresponding to any pump, register the same elevation.

If this condition is maintained with reasonable exactness during a test, the concentration of each sample will be a volume average for that sample during the test.

If the relative volumes of the samples which have been taken in this manner are used as weights for the corresponding samples, the weighted mean of all the samples will be the volume-average concentration for the test as a whole.

Finally, if all the samples thus taken are thoroughly mixed in one receptacle, or if given fractions of each of the samples are thus mixed, the concentration of the mixture will be the volume-average concentration for the test.

It may be added that similar procedure will serve for the method of weights, as against the method of volumes.

86.—Comparison of Discharges by the Method of Special Dilutions with Those Simultaneously Determined by the Method of Unbalanced Evaporations.—In these studies it has been the endeavor to develop a method which will eliminate as far as possible all the errors of the chemistry, as well as to discover the corrections to be applied in the chemistry where the method of unbalanced evaporations has been used.

Unbalanced evaporations result in the methods connected with such equations as (14), (17), (46), (53), (98), and (99). The corrections have been discussed in Section 43, *et seq.* The method of special dilutions applies particularly to such equations as (27), (28), (39), (44), (45), (56), (106), (107), and (108). It will strengthen the confidence in the method of correcting the errors for unbalanced evaporations and in the method of special dilutions, or balanced evaporations, if the results by these two methods agree.

In order to facilitate the comparison, Table 45 has been prepared. This not only enables a direct comparison in the individual tests, but also enables the direct comparison of the aggregates of discharge for all the tests on each unit, and all the tests collectively.

TABLE 45.—COMPARISON OF DISCHARGES OBTAINED BY THE METHOD OF SPECIAL DILUTIONS WITH THOSE BY THE METHOD OF UNBALANCED EVAPORATIONS.

UNIT NO. 11.			UNIT NO. 12.			UNIT NO. 13.		
Test No.	Discharge by special dilution.	Discharge by unbalanced evaporation.	Test No.	Discharge by special dilution.	Discharge by unbalanced evaporation.	Test No.	Discharge by special dilution.	Discharge by unbalanced evaporation.
105	1 443.4	1 448.8	200	1 564.5	1 571.0	23 *B*	1 398.9	1 421.8
106	1 539.0	1 538.5	201	1 515.7	1 515.0	26 *B*	1 497.4	1 474.1
107	1 519.4	1 518.9	202	1 481.1	1 470.1			
108	1 487.7	1 475.8	203	1 517.8	1 520.6			
109	1 492.0	1 482.4	204	1 505.3	1 505.0			
			205	1 512.2	1 511.0			
			206	1 491.4	1 493.5			
			207	1 485.4	1 488.0			
			208	1 501.8	1 496.2			
			209	1 560.0	1 561.7			
			210	1 539.4	1 543.2			
			211	1 514.1	1 545.9			
			212	1 489.4	1 492.9			
Sub-totals.	7 481.5	7 464.4	....	19 708.1	19 714.1	....	2 896.3	2 895.9

The aggregate discharge may be summarized as follows:

AGGREGATE DISCHARGE.

Unit No.	Special dilutions	Unbalanced evaporations.
11	7 481.5	7 464.4
12	19 708.1	19 714.1
13	2 896.3	2 895.9
Totals	30 085.9	30 074.4

The tests which are compared for Unit No. 11 were the first five which were run on this unit, and they were not so good as the later tests. No unbalanced evaporations, however, were made in the later tests, so that only five tests are available for the purpose of comparison. The discrepancy between the results by the two methods, nevertheless, is only 17.1 cu. ft. per sec. in about 7 500, or about $\frac{1}{7}$ of 1%, for the tests on Unit No. 11.

If the tests on Units Nos. 12 and 13 are compared collectively, the result is as follows:

AGGREGATE DISCHARGES OF UNITS NOS. 12 AND 13.

Special Dilutions.	Unbalanced Evaporations.
22 604.4	22 610.0

The difference is 6.4 cu. ft. per sec. in about 23 000, or a discrepancy of less than 0.03 per cent. In the case of the aggregates for all the tests, where both methods were used, the discrepancy is only slightly more than 0.02 per cent.

It may be concluded, therefore, that the method of correcting titrations in the case of unbalanced evaporations and the method of special dilutions are theoretically and practically correct.

87.—Determination of Size of Solution Tank, Sizes of Samples, and Strength of Silver Nitrate Solution.—In laying out a series of tests, it will be convenient to have simple formulas embracing the relations of titrations, volumes, and sizes of samples. As a general rule, it will be on the side of economy, as regards the capacity of the solution tank, to have the strength of the dosing solution as great as practicable. It was found quite satisfactory to have a salt solution of 300 grammes per liter, but as any increase in strength above this value adds considerably to the difficulty of dissolving the additional quantity of salt

required, it will generally be best to adhere to 300 grammes per liter, or less, as the occasion may seem to indicate.

As regards the concentration of the silver nitrate, it should be observed that the stronger this solution the more salt it will require to create a given titration on a sample of given size. Therefore, for economy of salt, it will be desirable to keep the nitrate solution as weak as consistent with definiteness of end points.

It follows from these remarks that computations involving the concentrations of the salt solution and silver nitrate should always have regard to the possible and practicable values of these quantities. The quantities which enter the equations are so related that there will be found a great variety of ways in which the tests may be planned, and this will be found very satisfactory, as it is rare that two series of tests on different installations will demand the same treatment.

Let e = salt equivalent of 1 gramme of silver nitrate = 0.344;

c = concentration of salt solution;

c_s = concentration of silver nitrate solution;

ρ = ratio of dilution of salt solution with distilled water;

t_1 = titration of a sample of normal head-water;

t_2 = titration of a sample of tail-water or special dilution;

V = nominal volume of a sample for titration after evaporation, or of dilute salt solution as pipetted into its casserole, that is, $V = V_1 = V_2 = V'_2$;

ρ_2 = ratio of sample volume measured for evaporation to the nominal volume to which the evaporation is carried before the titration; this also applies to ρ'_2;

R = ratio of dilution anticipated in any test;

v = titration per liter of salt solution based on the titration of a sample of dilute salt solution of the volume, V;

t = actual titration of the sample of dilute salt solution of the volume, V;

S = measured volume of a sample of tail-water, special dilution, or computed volume of a sample of normal head-water; that is, $S = S_1 = S_2 = S'_2$.

Then, the following relations are obvious:

$$e = 0.3441 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (171)$$

$$v = \frac{1\,000\,c}{c_s\,e} = \frac{2\,906\,c}{c_s} \quad \ldots\ldots\ldots (v \text{ in cubic centimeters}) \ldots\ldots (172)$$

Therefore, if c is the concentration of a sample of any kind, and t the titration, S being the volume of the sample, in liters, we have:

$$t = \frac{2\,906\, S\, c}{c_s} \text{ (general formula for titration)} \ldots\ldots\ldots (173)$$

Again,

$$v = \frac{\rho\, t}{V} \ldots\ldots\ldots (174)$$

so that

$$\frac{c}{c_s} = \frac{v}{2\,906} = \frac{\rho\, t}{2\,906\, V} = \frac{\rho\, \rho_2\, t}{2\,906\, S} \ldots\ldots\ldots (175)$$

$$c_s = \frac{871\,800\, V}{\rho\, t} = \frac{871\,800\, S}{\rho\, \rho_2\, t} (c = 300) \ldots\ldots\ldots (176)$$

From Equations (99),

$$R = \frac{v - k\, v_1}{v_2 - v_1} \ldots\ldots\ldots (177)$$

or

$$R = \frac{v}{v_2 - v_1} \text{ (approximately)} \ldots\ldots\ldots (178)$$

as v_1 is supposed to be small, relatively to v. Thus:

$$R = \frac{\rho\, \frac{t}{V}}{\frac{t_2}{S_2} - \frac{t_1}{S_1}} \text{ (approximately)} \ldots\ldots\ldots (179)$$

but $V = V_1 = V'_2$, $S = S_1 = S_2$, $t = t_2 = t'_2$, etc., so that

$$R = \frac{\rho\, \rho_2\, t}{t_2 - t_1} \ldots\ldots\ldots (180)$$

or,

$$\rho\, \rho_2 = R \left(1 - \frac{t_1}{t_2}\right) \ldots\ldots\ldots (181)$$

From this and an equation of the form of Equation (173),

$$\rho\, \rho_2 = R \left(1 - \frac{2\,906\, S_1\, c_1}{c_s\, t_2}\right) \ldots\ldots\ldots (182)$$

or

$$c_s = \frac{2\,906\, S_1\, c_1}{t_2 \left(1 - \frac{\rho\, \rho_2}{R}\right)} \ldots\ldots\ldots (183)$$

which, when multiplied by Equation (175), gives

$$c = \frac{\frac{\rho\, \rho_2\, c_1}{1 - \frac{\rho\, \rho_2}{R}}}{} = \frac{R\, c_1}{\frac{R}{\rho\, \rho_2} - 1} \ldots\ldots\ldots (184)$$

The foregoing equations, of course, assume that the method of special dilutions, or balanced evaporations and titrations, is to be used.

No normal head-water is evaporated, but the planning of the tests requires that the properties of the normal head-water shall be taken into account, so that, in any case, a series of analyses of the head-water should be made to lead to an accurate balance of evaporations in the actual tests.

The value of c_1, therefore, will be known in advance, having been determined from analyses made under circumstances closely simulating an actual test, and the value of S_1 may be chosen equal to S_2 in the foregoing equations. It is not likely that c_1 will be so variable as to render a close balance of titrations impracticable. The object of the calculations will not be frustrated by considerable relative variations in c_1, when they do occur.

Let it be required, for example, to plan a series of tests where the discharge will vary from 1 000 to 1 500 cu. ft. per sec. It may be observed that the ratio of dilution, R, and therefore, R', can be closely the same in all the tests by varying the rate of discharge, q, of salt solution so as to maintain

$$\frac{Q}{q} = R \text{ constant} \dots\dots\dots\dots\dots\dots \quad (185)$$

The values of $t_2 = t'_2 = t$, $S_2 = S'_2$, $V_2 = V'_2 = V$, and ρ, may be chosen arbitrarily, from which the values of c and c_s may be computed so as to satisfy all the requirements, it being merely necessary to see that c and c_s do not take on impractical values. When they do, a different choice of dimensions, capacities, titrations, or rates of dilution must be made, and repeated if necessary until satisfactory proportions are finally determined.

Therefore, let the data be taken as follows:

Evaporations are to be of $\frac{1}{2}$-liter samples to 10 c.c. in a casserole for titration, or $S = \frac{1}{2}$, $V = 0.01$, and $\rho_2 = 50$.

Titrations are to be limited to values less than 50 c.c., say $t = t_2 = 47.50$ c.c.

Dilutions of salt solution are to be 10 c.c. salt solution to 1 liter in a liter flask, 10 c.c. of the resultant mixture to be taken in a casserole for titration, or $\rho = 100$.

The concentration of the normal head-water has been previously determined to be 0.0133, as computed from a preliminary test of several salt-solution samples and special dilutions of the sort to be used in the tests.

The ratio of dilution may be first tried at $R = 6\,500$.

Then from Equations (183) and (184), we have

$$c_s = \frac{2\,906 \times 0.5 \times 0.0133}{(47.50)\left(1 - \frac{5\,000}{6\,500}\right)} = 1.76 \text{ grammes per liter} \quad \ldots\ldots (186)$$

$$c = \frac{6\,500 \times 0.0133}{\frac{6\,500}{5\,000} - 1} = 288.3 \text{ grammes per liter} \quad \ldots\ldots (187)$$

These concentrations, 1.76 grammes per liter for the silver nitrate and 288 grammes per liter for the salt solution, are good practical values, and satisfy the conditions. For a discharge of 1 500 cu. ft. per sec., the rate of discharge for salt solution must be

$$q = \frac{Q}{R} = \frac{1\,500}{6\,500} = 0.231 \text{ cu. ft. per sec.} \quad \ldots\ldots (188)$$

which is a trifle more than 6.5 liters per sec.

The salt rate, therefore, is

$$288.3 \times 6.53 = 1.880 \text{ kg. per sec.} \quad \ldots\ldots (189)$$

so that, if a test is to last for 15 min. with a margin of 10 min. before the start to secure steady conditions, there will be required

$$1.880 \times 60 \times 25 = 2\,825 \text{ kg. of salt per test} \quad \ldots\ldots (190)$$

which is equivalent to about 3.1 tons.

The total quantity of solution consumed per test would be $6.5 \times 1\,500 = 9\,800$ liters, or about 2 600 gal. This would probably demand a 3 000-gal. tank.

Now, the salt rate and rate of discharge of salt solution are, respectively,

$$\text{Salt rate} = q\,c = \frac{Q\,c}{R} = \frac{Q\,c_1}{\frac{R}{\rho\,\rho_2} - 1} \quad \ldots\ldots (191)$$

and

$$q = \frac{Q}{R} \quad \ldots\ldots (192)$$

It is clear, therefore, that the greater we take R, other things remaining the same, the less the salt requirement and the smaller the salt solution equipment.

The value of R, however, is limited, to a large extent, by the degree of accuracy required in the test. In order to secure a given degree

of accuracy, for example, it will be necessary to introduce a certain quantity of salt into the head-race to secure a sufficiently large concentration. This will be required, as the titration must not be less than a certain quantity when there is a limit of permissible error.

Perhaps the following is a better procedure than that of the foregoing problem. Suppose the degree of accuracy requires that errors of 1 in 400 shall be easily appreciable. Then, as it is easy to detect 0.1 c.c. on the scale of a 75-c.c. burette, it may be required that sufficient salt, over and above that contained initially in the head-water, shall be introduced to require 40 c.c. of silver nitrate in the titration. The concentration of the salt solution will be 300 grammes per liter, and that of the silver nitrate will be 1.5 grammes per liter. The former is about as strong as it can be taken, and the latter is about as weak as practicable, according to the investigations of Dr. Mellet. The ratio of dilution, $R = R'$, is given by

$$R = 2\,906 \frac{c}{c_s} \frac{S}{t_2} \qquad (193)$$

where t_2 is not the titration of the tail-water, but is that part of it due to the introduction of salt. Thus, $t_2 = 40$ in this case. S is limited by the quantity which may be conveniently taken for evaporation, but should evidently be as large as this limit. A quantity of $\frac{1}{2}$ liter is about as large as most cases will admit. A liter makes a long evaporation and concentrates to a greater extent all the other impurities of the water along with the salt. However, when the method of special dilutions is used, it will be found that liter-evaporations, and even larger, will be possible when the discharge to be determined is very great.

Hence, with $\frac{1}{2}$-liter samples,

$$R = 2\,906 \times \frac{300}{1.5} \times \frac{0.5}{40} = 7\,265.$$

By Equation (184) we find

$$\rho = \frac{R}{\rho_2 \left(1 + R \frac{c_1}{c}\right)} \qquad (194)$$

which, for the foregoing determined value of R, and $\rho_2 = 50$, $V = 0.01$, $c_1 = 0.0133$, becomes

$$\rho = \frac{7\,265}{50\,(1 + 0.322)} = 109.7.$$

Thus we find that

$$1 - \frac{\rho\,\rho_2}{R} = 1 - \frac{109.7 \times 50}{7\,265} = 0.245$$

and hence, by Equation (183),

$$t_2 = 2\,906 \quad \frac{S}{\left(1 - \frac{\rho\,\rho_2}{R}\right)} \frac{c_1}{c_s} \dots\dots\dots\dots\dots (195)$$

which, on substitution of values, gives $t_2 = 52.9$, and thus checks the value 12.9 of t_1, given by Equation (173), on addition of 40 to the latter.

The rate of discharge of the salt solution is

$$q = \frac{Q}{R} = \frac{1\,500}{7\,265} = 0.206 \text{ cu. ft. per sec.,}$$

which is the equivalent of 5.8 liters per sec., when the discharge to be measured is 1 500 cu. ft. per sec.

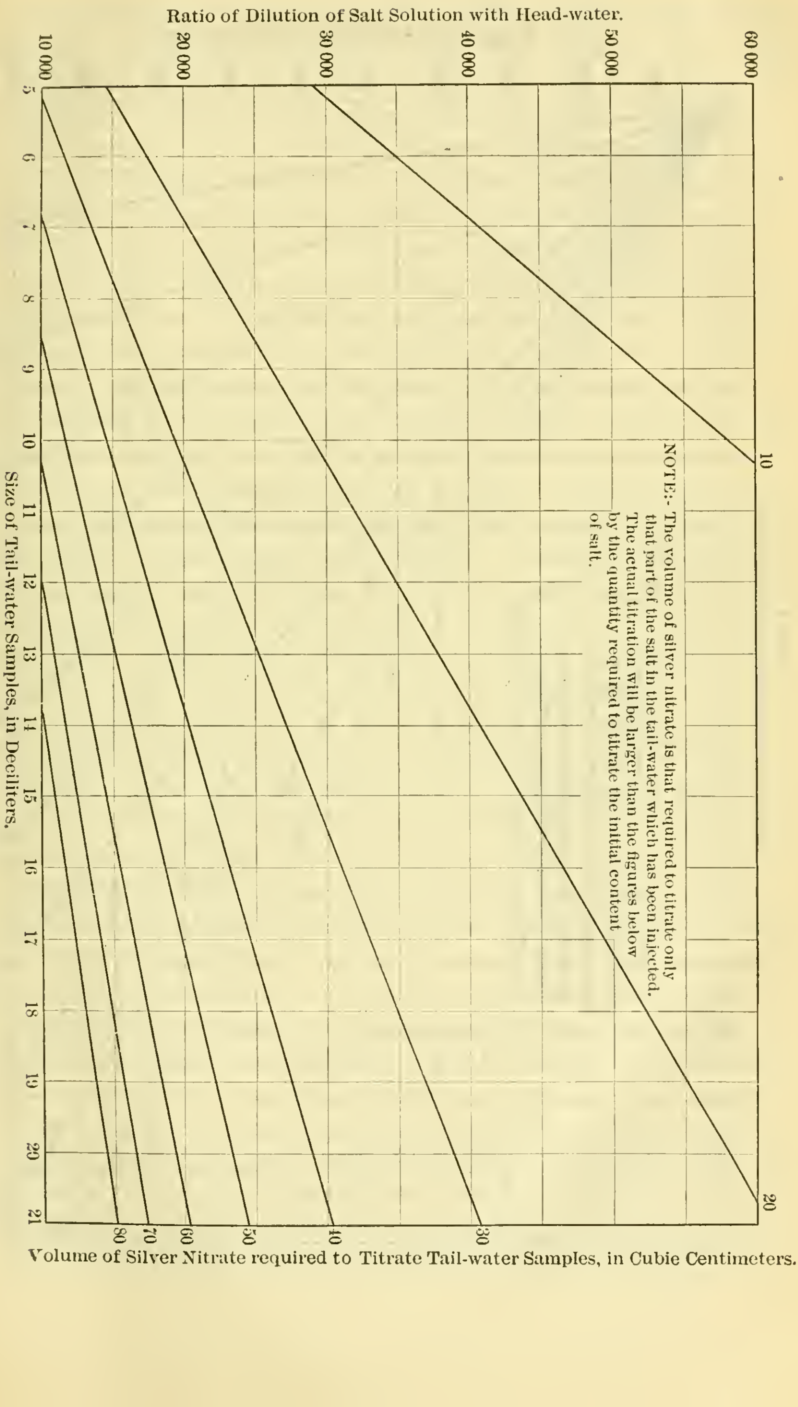
The size of the salt-solution tank for a 15-min. test, with 10-min. discharge prior to the start to secure steady flow, is $25 \times 60 \times 5.8 =$ 8 700 liters, and this would require $8\,700 \times 0.300 = 2\,610$ kg. of salt, or about 2.9 tons of salt for each test.

In practice it would probably be better to take $R = 7\,225$, which is the square of 85, and $\rho = 110$. Thus the special dilutions can be quickly made with two 850-c.c. flasks and a 10-c.c. pipette; and the dilution with distilled water can be effected in a flask containing 1 100 c.c., the actual value of ρ having to be computed from the flask and pipette calibrations in any case where precision is to be exacted.

Figs. 7 and 8 will facilitate the determination of the relations between ratio of dilution, size of sample, and titration of the injected salt when the ratio, $c \div c_s$, of the concentration of the salt solution to that of the silver nitrate is 200, which corresponds to concentrations of 300 and 1.5, respectively.

*88.—Care of Samples, and Method for Taking and Treating.**— These operations are among the most important connected with the turbine tests. Consequently, the chemists engaged in this work must take the most scrupulous care to see that samples are properly taken, and not allowed to become contaminated before they are treated in the chemical laboratory.

* Sections 88 and 89 are copied from the original instructions to chemists, with slight modification. They pertain particularly to the method of measuring samples by volume.



Ratio of Dilution of Salt Solution with Head-water.
10 000
20 000
30 000
40 000
50 000
60 000
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
Size of Tail-water Samples, in Deciliters.
10
20
80
70
60
50
40
30
NOTE:- The volume of silver nitrate is that required to titrate only that part of the salt in the tail-water which has been injected. The actual titration will be larger than the figures below by the quantity required to titrate the initial content of salt.
Volume of Silver Nitrate required to Titrate Tail-water Samples, in Cubic Centimeters.

FIG. 7.

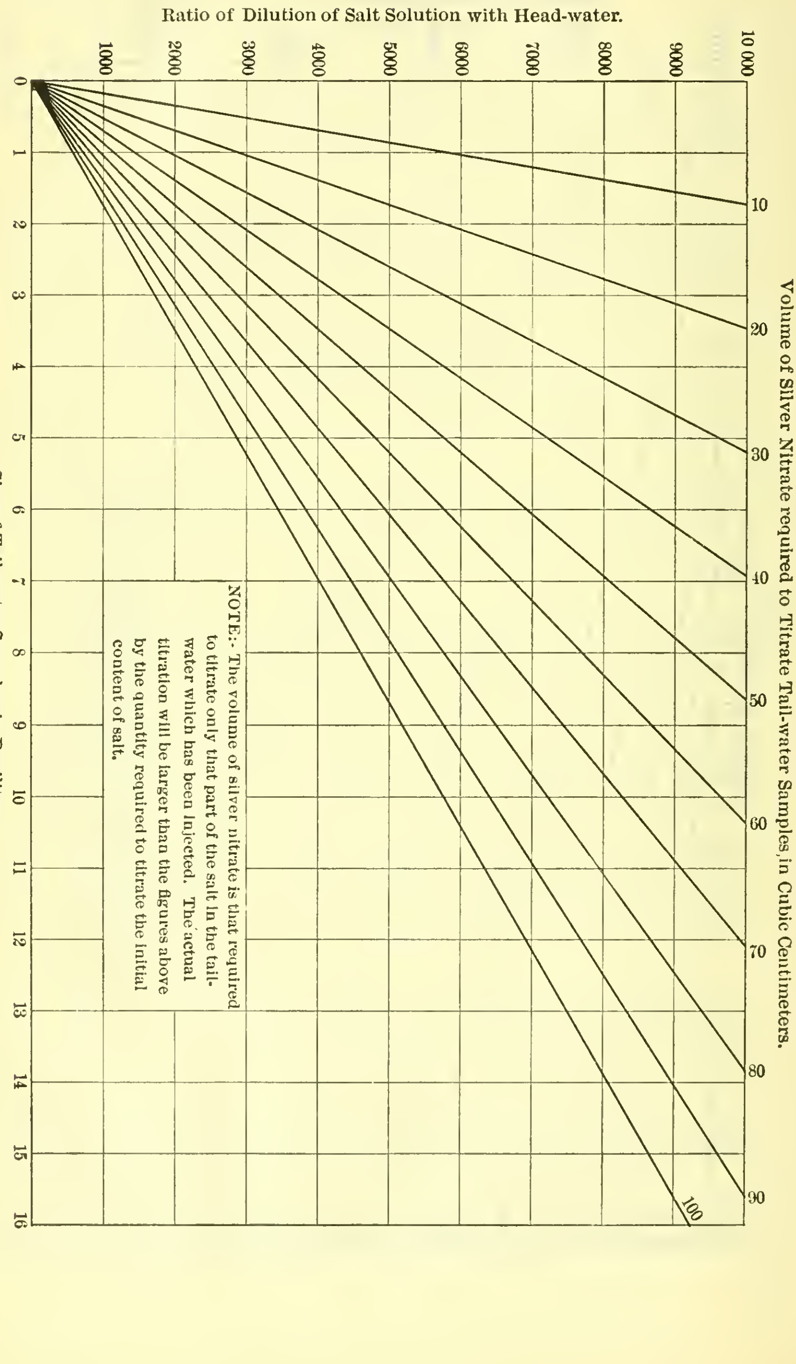

Ratio of Dilution of Salt Solution with Head-water.
1000
2000
3000
4000
5000
6000
7000
8000
9000
10 000
Volume of Silver Nitrate required to Titrate Tail-water Samples, in Cubic Centimeters.
10
20
30
40
50
60
70
80
90
100
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Size of Tail-water Samples, in Deciliters.
NOTE:- The volume of silver nitrate is that required to titrate only that part of the salt in the tail-water which has been injected. The actual titration will be larger than the figures above by the quantity required to titrate the initial content of salt.

FIG. 8.

Three classes of samples are taken during each test: The first is the salt solution sample, which is taken from the 3-in. supply pipe leading from the salt solution tank to the suction of the 8-in. centrifugal pump which feeds the twelve distributing pipes. The second is the sample of the normal canal water or normal head-water taken from the head-race before the water reaches the distributing pipes. The third is the sample of the tail-water after it has been dosed with the salt solution and after it has passed through the racks, turbines, and draft-tubes. These latter samples are taken from eighteen* small pumps which are in the chamber immediately above the draft-tubes of the turbine under test. It is imperative that a chemist be in charge of each of these three classes of samples. The chemist in charge of any one of these classes is to keep his sample bottles in view at all times from the time that he leaves the chemical laboratory until he returns thereto with the samples. The object of this is to prevent the possibility of the samples becoming affected in any way through the breakage of a bottle, the accidental introduction of extraneous substances or the accidental interchange of two different samples. In order to render this care easy and systematic, a number of carrier boxes, provided with compartments, have been made, and each compartment must be numbered so that the corresponding sample of the same number will always be placed in that particular compartment. In the same way the draining racks in the laboratory are to be numbered, and bottles containing the samples of the corresponding numbers are always to be drained in the rack carrying the particular number which pertains to that bottle. The openings on the water baths shall also be numbered in the same manner, so that the same sample is always evaporated on the same opening and supplied from the same separatory funnel in the funnel rack. In similar manner there shall be provided a table, in the immediate vicinity of the water baths, carrying squares corresponding in position to the openings in the water baths, and these squares shall be numbered in a similar manner to the openings on the water baths. In order still further to systematize the handling of these samples, it may be understood that odd numbers pertain to the west side of the head- and tail-races, and even numbers to the east side. It is a simple matter then to arrange the numbers in rows with the odd numbers representing the west side and the even numbers

* Originally there were only twelve regular tail-water samples.

the east side. After a complete set of samples has been taken during any test, the bottles are to be tightly corked and carried to the laboratory in charge of the particular chemist whose duty it is to care for the particular class of samples in question.

Normal Head-Water Samples.—The duty of the chemist who is in charge of the normal head-water samples shall be: first, to ascertain the length of the time of the test; second, to ascertain the positions from which normal head-water samples are desired; third, to record the time of beginning and the time of ceasing to take the normal head-water samples. These times shall be recorded by the chemist and turned in with his record of observations during the test. Ordinarily, one normal head-water sample will be taken from each of the four normal head-water sampling pumps during each test. Each sample is to contain approximately 1 gal. The odd-numbered samples shall be taken from the west sampling pipes, and the even-numbered samples from the east sampling pipes; and on every record the position and depth of the intake of the suction pipe on these sampling pumps during the test shall be noted by the chemist in charge of the particular samples in question. At the end of the test these samples are to be carried to the laboratory in the view of, and under the direction of, the chemist in charge. It is an absolute rule in taking these samples that every endeavor be made by all parties concerned to see that the sample fills into the proper sample bottle at as nearly a uniform rate as possible.

Tail-Water Samples.—The chemist in charge of the tail-water samples shall first ascertain the length of time of the test and the time of beginning. He shall then see that the tail-water sampling pumps are started in sufficient time to permit him to gauge the discharges, so that his gallon sample bottles will be nearly, though not quite, filled, during the test. He is then to proceed from the laboratory to the chamber immediately above the draft-tube of the turbine to be tested, where the sampling pumps are placed. The eighteen sample bottles are then to be placed on the movable boards provided for sliding the bottles under the proper sampling pipes. It shall also be the duty of this chemist, as in the case of all other chemists in charge of sampling bottles, to see in advance that each bottle is properly cleaned and prepared for receiving the sample. The greatest care should be taken to see that bottles are thoroughly rinsed with the dosed tail-water and thoroughly drained before admitting the samples. He shall also keep

the samples in his view at all times, so that there will be no possibility of contamination or interchange of samples by accident or otherwise. A few minutes before the beginning of a test, a long ring of the telephone bell will indicate to this sample man that the test is about to begin. A short time later three long rings of the bell will indicate the instant of beginning the test. The test shall begin at the instant when the third long ring begins. One minute after receiving this signal the levers controlling the parts which carry the sampling bottles shall be moved over into such a position that all the bottles will begin to receive their sample at that instant. A few minutes before the end of the test a long ring on the telephone bell will indicate that the test is about to end. Simultaneously at the beginning of the third of the three long rings which follow the preliminary long ring, the test shall be at an end, and 1 min. thereafter the levers controlling the boards carrying the tail-water samples shall be moved in such a manner that the bottles will cease to receive any water from the sampling pipe at that instant. The chemist in charge of the tail-water samples shall then proceed as quickly as possible to have his samples carefully carried to the chemical laboratory and properly cared for thereafter.

Salt Solution Samples.—These samples shall be taken from the 3-in. pipe leading from the salt solution tank to the suction of the 8-in. centrifugal pump at a point as near as practicable to the suction pipe. There shall be five of these 1-qt. samples, marked A, B, C, D, and E. It shall be the duty of the chemist in charge of these samples: first, to ascertain the time of duration of the test; second, to divide that time into five equal parts; and third, to see that the samples are taken in sequence, beginning with A and ending with E, each one being held under the sample cock for one-fifth of the duration of the test. It shall also be his duty to see that the sampling cock gives a uniform discharge during the taking of any one sample and that this rate of discharge is such as nearly to fill the bottle during the particular time devoted to the particular sample in question. All the other requirements for cleanliness and nicety in the work connected with sampling shall be observed by all the chemists, in order to secure the desired degree of accuracy for the turbine tests. In particular, all sample bottles, after having been previously cleaned, shall be rinsed with a portion of the sample water before being filled with another portion which is to constitute the sample proper.

Reagents.—There are three principal reagents connected with the turbine tests. The first is the commercial salt, which has been purchased in large quantity, for the purpose of making the salt solution. This salt contains more or less sand, also pieces of paper, chips of wood, and other extraneous substances, which are largely screened out during the process of charging the salt-solution tank. About 12 000 lb. are to be dissolved in the 5 000-gal. tank provided for the purpose. The resulting solution is then injected into the head-water through the twelve distributing pipes. The object of these distributing pipes is to assist in getting a good mixture of the salt solution with the feed-water for the turbine.

Silver Nitrate.—The second reagent used in this work is the centi-normal, or approximately centi-normal, solution of silver nitrate. Although it is not necessary that this solution should be standardized with the highest degree of accuracy, it is desirable that some degree of precision be required in determining the weight of silver nitrate dissolved per liter of distilled water. This is for the purpose of securing uniformity of strength from day to day. The general procedure for making the silver nitrate solution shall be as follows: About 14.5 grammes of silver nitrate shall be dissolved in 1 liter of distilled water. In weighing this quantity of silver nitrate for this purpose, the margin of error allowed in the weight may be taken at 0.05 gramme. After this solution is made to the extent of 1 liter in a dark-colored bottle, it shall be carefully preserved in a covered box where it will be entirely protected from the light except at short intervals when the solution is to be taken from the bottle. The solution required in the titration is to be made up from this solution at various times when needed, by diluting 100 c.c. to 1 liter of distilled water. In case larger quantities are needed, correspondingly larger quantities may be diluted.

It is an absolute rule, in reducing tests in the laboratory, that the same silver nitrate solution be used during any one particular test, as this rule is imperative in order to eliminate the errors of titration due to the excess of silver nitrate required to give a definite end reaction and otherwise to secure precision.

Chromate of Potassium.—Chromate of potassium shall be used as an indicator. This solution shall be made by dissolving 50 grammes in 1 liter of water, or, if larger or smaller quantities are desired, in corresponding proportions.

Quality of Reagents.—In the case of all the foregoing reagents, except the salt, chemically pure chemicals shall be used.

Titrations.—All the samples are generally taken and treated in such a way that the titration will require about 50 c.c. of silver nitrate. It will be necessary, therefore, to use 100-c.c. burettes, as, in many cases, the titration may exceed 50 c.c. Further than this, the laboratory procedure reduces each sample, either by dilution or evaporation, to about 10 c.c. in a casserole, so that the titrations of all samples are nearly identical in character, as the samples contain nearly the same quantity of chlorides at approximately the same concentration. Wherever possible, the method of balanced evaporations, Section 45, shall be used in concentrating by evaporation.

In order to secure uniformity of treatment, several rules must be strictly observed by the chemists when titrating samples.

Procedure.—

1.—The burette shall be filled to the zero of the scale, as nearly as it is practical without loss of time, say to 0.4 or 0.3 c.c. Care must be taken to scrape off all surplus solution from the discharge orifice of the burette with a clean glass rod and to see that there is no surplus solution clinging to the top of the burette tube on the inside where it would be likely to fall into the solution below, thus causing an error in the estimate of the quantity of nitrate used in the ensuing titration.

2.—Take a reading of the burette.

3.—Record this reading on the form for the purpose, also noting the number of the burette beside this reading.

4.—Place the casserole containing the samples to be titrated under the burette.

5.—Introduce one drop of indicator into the sample by using a dropping bottle.

6.—One minute, approximately, after the first reading of the burette, take a second reading and record it immediately under the first. In general, these readings will be found to agree. If not, wait another minute, take a third reading, and record it immediately under the second. The third reading should agree with the second. If not, continue to take and record readings one minute apart until two consecutive readings, taken in this manner, agree.

7.—The stop-cock of the burette is now to be adjusted so that about two drops per second of silver nitrate will fall into the sample being

titrated. The burette must be adjusted in the support so that the drops will not fall far enough to cause any splashing of the solutions, thus preventing losses over the side of the casserole.

8.—While the silver nitrate is thus being discharged into the sample, the observer has time to rinse a glass stirring rod with distilled water and to stir the sample with it in order to secure a thorough mixture. meantime watching the level of the silver nitrate solution in the burette and keeping the stop-cock adjusted so as to maintain the rate of discharge at about two drops per second.

9.—For each 10 c.c. of silver nitrate admitted to the sample in titration, one drop of chromate of potassium indicator is to be added to the sample as the level of the silver nitrate solution falls past the corresponding graduation mark on the burette. The object of this is to keep the quantity of indicator present in the sample proportional to the quantity of silver nitrate consumed, and also proportional to the volume of solution in the casserole. The color of the sample is also thus preserved as nearly uniform during the titration as possible.

10.—As soon as the titration nears the end point, the orange tint which appears on the admission of each drop of nitrate will tend to persist a longer time as the solution is stirred continuously with the glass rod. A little experience with samples of a particular class will enable the observer to tell when the titration is within about 2 c.c. of the end.

11.—When the titration is thus brought within 1 or 2 c.c. of the end, the observer must readjust the stop-cock of the burette so as to retard the rate of the discharge of silver nitrate to about one drop in 2 or 3 sec., stirring the solution thereafter continuously and scrutinizing the changing tints of the solution closely. Finally, a drop of silver nitrate will fall into the casserole which causes the solution to take a faint, though perfectly definite, orange tint. When this occurs the observer must be prepared to close the stop-cock instantly so as to stop the discharge from the burette. On continuing the stirring for 30 or 40 sec., the orange tint may gradually fade out, giving place to the former lemon-yellow tint. If this is the case, add one more drop of silver nitrate from the burette, and continue the stirring. By pursuing this course, a drop from the burette will be finally admitted into the casserole which will bring the solution to a very faint but definite and permanent orange tint. This is the end point sought. Care must

be taken before reaching this stage to see that there is no surplus drop of silver nitrate clinging to the bottom of the burette after reaching the end point, otherwise the burette reading will be erroneous, and an error will have been committed. The best safeguard at this stage is to remove each drop as it forms at the tip of the burette with the end of the glass stirring rod and introduce it in this way into the solution, closing the stop-cock each time before removing the drop.

12.—Read the burette scale and record the reading on the form for the purpose. A minute later check the reading and indicate the check by a check mark opposite the last reading.

If the solution has not been drawn from the burette faster than permitted by these rules, it will rarely be found that a third reading at the end of the titration is necessary.

13.—The observer is now to compute the number of cubic centimeters of silver nitrate consumed in the titration. After checking his calculation carefully, he is then to affix his initials opposite the result, and proceed to another titration, the sample for which shall be selected by the rules for group titrations, Section 48.

Preparation of Salt Solution Samples.—

1.—Shake the bottle containing the sample of salt solution to be treated.

2.—Take a clean 10-c.c. pipette, rinse it with a portion of this sample, allowing this portion to be discharged into a sink or waste receptacle.

3.—Fill the pipette again from the sample and withdraw very slowly from the solution so as to drain the outside of the pipette thoroughly.

4.—Let the sample in the pipette sink slowly to the graduation mark in the usual manner and, if necessary, scrape off the surplus drop at the tip with a clean dry glass rod.

5.—Also examine the tip and outside of the pipette to see if there is any surplus solution which can in any manner become incorporated by accident with the solution to be delivered by the pipette. If so, it must be removed, being careful not to contaminate the contents of the pipette.

6.—Discharge the pipette into a liter flask, which has been previously cleaned and rinsed with distilled water. The neck of this flask must be free from drops of distilled water when the salt solution is

introduced. The manner of discharging will depend on the manner of calibrating the pipette by the chemists.

7.—Fill the flask to the graduation mark with distilled water, and make a perfect mixture by stopping the bottle and inverting it from thirty to forty times, giving it a shake each time and allowing the air to pass from top to bottom and bottom to top after each inversion.

8.—Take the temperature of both the salt solution and the distilled water immediately after the sample, or samples, have been diluted as just described.

9.—Record these temperatures on the forms provided for the purpose, and observe whether the level of the solution in the flask when held vertical is still at the graduation mark.

10.—A 10-c.c. pipette full of the dilution just described delivers into a casserole a sample of dilute salt solution ready for titration. Chemists must observe the rules already given for the discharge of pipettes.

11.—Take and record the temperature of the dilution in the liter flask immediately after charging all the casseroles which are to contain samples for titration from this flask.

12.—Empty all flasks and other receptacles containing solutions for which there is no further use.

Normal Head-Water Samples.—These samples are of the normal canal water taken in the head-race before the water receives the salt solution from the distributing pipes. Four 1-gal. sample bottles are nearly filled with this water. Dilutions of salt solution with portions of these samples are prepared for each test. As a general rule, only one rate of dilution will be used in reducing any given test.

Special Dilutions.—Several different rates of dilution, however, may be used in preparing the normal head-water samples for titration. Three examples follow. Observe carefully the general rule for making accurate dilutions, Section 39.

Dilution A.—Take five 10-c.c. pipettes of salt solution and dilute with 3 liters of normal head-water, thus making approximately 3 050 c.c. of stock solution in a gallon bottle. Shake thoroughly.

A_{123}.—One of these 3 liters for the dilution must come from each of the bottles containing normal head-water samples 1, 2, and 3. Label the stock solution thus prepared A_{123}.

A_{234}.—In precisely similar manner prepare another stock solution, taking 1 liter each from normal head-water samples 2, 3, and 4. Label this stock solution A_{234}.

Dilution B.—Take four 10-c.c. pipettes of salt solution and dilute with 2½ liters of normal head-water, thus making approximately 2 540 c.c. of stock solution in a gallon bottle. Shake thoroughly.

B_{12}.—One of these 2½ liters for the dilution must come from normal head-water sample bottle 1 and the remaining 1½ liters from bottle 2. Label this stock solution B_{12}.

B_{34}.—In similar manner prepare another stock solution, taking 1 liter from normal head-water sample 4 and the remaining 1½ liters from normal head-water sample 3. Label this stock solution B_{34}.

Dilution C.—Take three 10-c.c. pipettes of salt solution and dilute with 2 liters of normal head-water, thus making approximately 2 030 c.c. of stock solution in a gallon bottle. Shake thoroughly.

C_{12}.—One of these liters for the dilution must come from each of the bottles containing normal head-water samples 1 and 2. Label this stock solution C_{12}.

C_{34}.—In similar manner prepare another stock solution, taking 1 liter from each of the bottles containing normal head-water samples 3 and 4. Label this stock solution C_{34}.

89.—To Make Accurate Dilutions with the Chemical Equipment in the Laboratory.—Let it be required to dilute n pipettes of salt solution with m liter flasks of solution which will be called water, the whole to be mixed in a larger receptacle. Use only flasks which contain a given quantity of water, nominally 1 liter. Divide as nearly as possible the n pipettes of salt solution among the m flasks, discharging them into the flasks according to the rules for pipettes, and preparing the flasks according to the rules for flasks.

Fill the flasks to their graduation marks with water and shake thoroughly, according to the directions already given. Let the flasks rest on a table for a few minutes and then, if necessary, add a drop or two more of water to restore the level of the meniscus to the graduation mark.

Empty all the flasks into the receptacle which has already been cleaned, rinsed with distilled water, and drained, as previously directed for flasks. Shake thoroughly, empty the contents, and repeat the entire

procedure, except that the final mixture in the receptacle is to be retained as the desired mixture.

The total contents of the flasks divided by the total discharge of the pipettes is the ratio of dilution.

In case it is desired to make a dilution up to a given ratio of mixture, the rule is correspondingly the same as the foregoing except that the n pipettes are discharged into the receptacle instead of the flasks, after which the m flasks of water are poured in. The flasks are finally rinsed into the receptacle with the final mixture therefrom. The resulting ratio of mixture is the total contents of the m flasks divided by the total discharge of the pipette, or pipettes, as the case may be. When n is not a multiple of m it will be on the side of precision to dilute for ratio of mixture.

Further Treatment of Special Dilutions.—In every test, therefore, there will be two stock bottles containing the special dilutions or stock solutions. These special dilutions are to be further diluted by discharging a 10-c.c. pipette of the special dilution into a liter flask which has been previously cleaned and rinsed with normal head-water. The flask is then filled with normal head-water in the usual manner and the stopper tightly adjusted in the neck. The flask must then be alternately inverted and righted forty times, allowing the air bubble to rise to the highest point within the flask on each half turn. When satisfied that the mixture is uniform, the sample is to be divided into two half-liter samples by pouring it into two half-liter flasks which have been previously rinsed with distilled water, the liter-flask containing the original sample being finally rinsed with distilled water, which may be divided between the two half-liter flasks. Each of these half-liter samples is then to be poured into the particular separatory funnel which has been reserved for it. The flask is then to be rinsed with a small portion of distilled water which is finally poured into the separatory funnel.*

Prepare ten such half-liter samples altogether, taking four from one of the stock bottles of special dilution and the remaining six from the other stock bottle. Mark each casserole, with a label attached to the handle, with the symbol which indicates the particular stock

* This procedure was adopted so as to make effective use of the particular glassware in the laboratory.

bottle out of which the sample to be evaported in the casserole was taken.

These samples are next to be evaporated over the water-bath, the attendant maintaining a volume of about 40 c.c. in each casserole during the evaporation. This is effected by drawing from the separatory funnel over each casserole at proper intervals of time sufficient solution to restore the level of the liquid in the casserole to the proper elevation.

When all the sample which will discharge from any one funnel into its casserole has been drawn off in this manner, the attendant must rinse down the sides of the funnel with a small quantity of distilled water which is also admitted to the casserole at the proper time.

Great care must be taken in all the operations connected with these evaporations to see that no part of any sample is lost, and that all the salt of any sample is finally collected in the proper casserole, also to see that the level of the solution in every casserole is maintained as nearly fixed as reasonable care can insure.

When all the salt from any sample is collected in this manner, there should be about 40 c.c. of solution in the casserole. The evaporation is then to be continued until the volume of the liquid in the casserole is reduced to about 10 c.c.

This can be accurately estimated by the attendant by comparison with the volume of water, in another casserole of the same size and shape, which results from the delivery of a 10-c.c. pipette.

The casserole containing this final concentration of salt is then to be placed on the square reserved for this purpose on the sample table.

This sample is then ready for titration.

Tail-Water Samples.—There are eighteen of these samples, taken in gallon bottles from eighteen different locations in the tail-race. The numbers are not consecutive, being, in particular, 21 to 32, both inclusive, 41, 42, 43, 51, 55, and 56. This irregularity in numbering resulted from a series of changes which were made in the locations of the tail-race sampling pipes.

Evaporation of Tail-Water Samples.—One-half liter is measured from each of the eighteen samples into the corresponding separatory funnel, the half-liter flask in each case being rinsed with a small portion of the sample before being filled with it and with a small portion of distilled water after being emptied, this portion of distilled

water finally being poured into the separatory funnel to prevent any loss of salt.

The evaporation is then to be conducted in precisely the same manner as that given for the evaporation of the special dilutions.

Modified Procedure for Tail-Water Samples.—It is desirable in some cases to reduce the number of evaporations of tail-water samples. In order to make the process clear, it will aid to observe that the number and relative location of each sample in the tail-race is indicated by the following diagrammatic tabulation:

Upper Draft-Tube (Looking up stream).

	WEST.			EAST.	
43		29		55	
	23		32		56
51		31		41	

Lower Draft-Tube (Looking up stream).

	WEST.			EAST.	
	25		28		22
21		27		26	
	42		30		24

The samples, therefore, may be classed according to the particular half of the draft-tube to which they belong, thus:

Upper Draft-Tube.		43		29
	WEST HALF		23	
		51		31
			55	
	EAST HALF	32		56
			41	
Lower Draft-Tube.			25	
	WEST HALF	21		27
			42	
		28		22
	EAST HALF		26	
		30		24

The samples represent approximately equal areas of cross-section of the draft-tube. The samples of any half of a draft-tube, therefore, will be combined by mixing equal parts, say ½ liter, of each in a gallon bottle which has been previously cleaned and then rinsed with a mixture of approximately equal portions of the corresponding samples.

Thus, for the west half of the upper draft-tube, mix ½ liter of each of Nos. 43, 51, 23, 29, and 31. For the west half of the lower draft-tube, mix ½ liter of each of Nos. 21, 25, 42, and 27.

Shake the resulting stock solutions, consisting of 2 or 2½ liters, in gallon bottles thoroughly, and label the bottles according to the following scheme.

Upper Draft-Tube, West Half..............	U. W.
Upper Draft-Tube, East Half..............	U. E.
Lower Draft-Tube, West Half..............	L. W.
Lower Draft-Tube, East Half..............	L. E.

Measure accurately from each stock solution two ½-liter samples which are to be placed in the separatory funnels reserved for the particular samples in question.

In all cases the volumetric flasks must be cleaned and rinsed with a portion of the stock solution from which the sample is to be taken before being filled with the sample. Moreover, the flask from which any sample has been discharged into its separatory funnel must be finally rinsed with a small portion of distilled water, which portion must then be discharged into the separatory funnel, so as to insure the capture of all the salt of the sample.

Each casserole must finally be labeled with the symbol which pertains to the particular stock solution which is to be evaporated in it.

In all cases the same separatory funnel and location on the water-bath should be reserved for the same sample in different tests.

PART III.

THE TESTING PLANT.

SUMMARY.

90.—In Part III the general plan for the salt solution tank and distributing system is described.

The current-meter rack and platform, which also carries the salt solution distributing pipes, is described and illustrated. This structure with its equipment can be moved bodily by the crane from one unit to another.

The salt dissolving box and platform is illustrated. By this invention 7 tons of commercial salt can be put into solution in a 5 000-gal. tank within 1 or 2 hours, according to circumstances. Its operation is continuous. The solution is very nearly saturated.

The sprinkler system is described and illustrated.

The large iron calibrator, or pipette, is described.

The tail-race sampling pumps are illustrated and the method of operating them explained.

The chemical laboratory and equipment and the direct-current metering instruments are illustrated.

As the method of measuring the discharge of hydraulic turbines by a chemical introduced into the feed-water is a relatively new branch of engineering, especially when applied to hydro-electric units of large capacity, a somewhat detailed description of the chemical testing plant, as it was installed for the tests considered herein, will be necessary, in order to make clear the exact nature of the operation of testing as it was conducted by the writer. As a matter of fact, the methods were entirely novel in nearly all respects.

The magnitude of the chemical operations conducted may be understood better when it is explained that each test required from 4 to 6 tons of commercial salt, which was used as a chemical reagent. The problem of getting this quantity of salt into a solution very near to the saturation point alone offered considerable apparent difficulty at the outset. Numerous preliminary tests were made to ascertain the probable length of time which would be required to make such a solution in a 5 000-gal. tank, with the apparent result that it would require several hours and great care. It was apparently very difficult to make

a saturated solution of 1 liter capacity in a short period of time. On careful reflection, however, it was decided to make what the writer terms a dissolving box, and this invention proved to be very efficient in producing salt solutions nearly up to saturation. Moreover, it was found by actual test that 7 tons of salt could be dissolved and the resulting solution pumped into a 5 000-gal. tank in less than 1 hour.

In the case of tests which had been conducted in Europe, on turbines discharging a very small quantity of water, it appears to have been the custom to filter the salt solution before admitting it to the solution tank. This appeared to the writer to present too many chances for delay, and, therefore, it was decided to try out a method which would not demand the additional time necessary for filtration.

91.—General Plan for Salt-Solution Tanks and Distributing System.—Fig. 9 is the writer's original diagrammatic sketch for the salt solution tanks and distributing system. Very little change was made in this general plan when laying out the details of the actual system as it was installed. On the left is the dissolving box which is substantially of the same dimensions as the one actually used. Instead of a 3-in. pipe, however, leading to the salt solution screens, as shown in the sketch, a trough about 8 by 10 in. in cross-section led from the top of the dissolving box on a steep slope to a point where its discharge was emptied into a hood fitting over the end of the trough, hanging vertically therefrom and provided with screens of very fine mesh in the bottom. This hood could be taken from the trough at any time for the purpose of emptying the accumulations of rubbish. The latter consisted principally of chips of wood and other extraneous matter which was included as impurities of the low commercial grade of salt used for the tests. At several points in the trough there were screens of somewhat coarser mesh than those at the bottom of the hood. These were slipped into slots in the sides of the trough and served to remove the coarser impurities without allowing them to fall into the hood. The salt solution, after passing through the screen at the bottom of the hood, fell into a 5 000-gal. tank. It was originally intended to have a 6 000-gal. tank, but the one supplied contained only 5 000 gal. and appeared to fulfill all necessary requirements. The hood into which the salt solution fell from the trough was approximately 12 in. wide, 15 in. long, and 18 in. high; the screen therefore

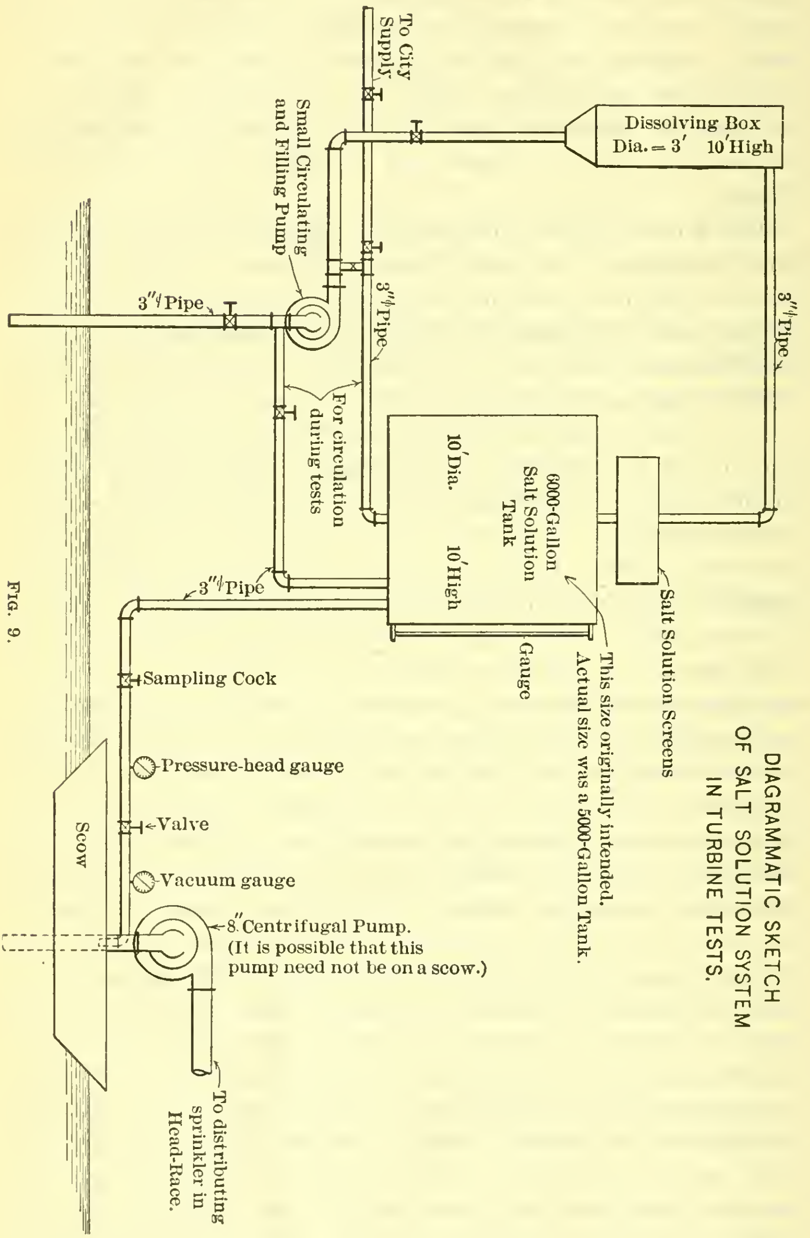

DIAGRAMMATIC SKETCH
OF SALT SOLUTION SYSTEM
IN TURBINE TESTS.
To City
Supply
Dissolving Box
Dia. = 3′ 10′ High
Small Circulating
and Filling Pump
3″ Pipe
3″ Pipe
3″ Pipe
For circulation
during tests
6000-Gallon
Salt Solution
Tank
10′ Dia.
10′ High
Salt Solution Screens
This size originally intended.
Actual size was a 5000-Gallon Tank.
Gauge
3″ Pipe
Sampling Cock
Pressure-head gauge
Valve
Vacuum gauge
Scow
8″ Centrifugal Pump.
(It is possible that this
pump need not be on a scow.)
To distributing
sprinkler in
Head-Race.

Fig. 9.

offered an effective area of about 12 by 15 in. This screen had forty meshes to the inch.

At the side of the salt-solution tank there was a 10-ft. gauge-board, having a vertical slot along the axis of the board sufficiently deep and wide to admit a glass tube of 1 in. inside diameter, the bottom of which was connected to the bottom of the tank by a short length of 1-in. rubber tubing. There were also two valves, one being at the bottom of the glass tube for the purpose of emptying the gauge glass in order to admit fresh solution from the tank. The object of this operation was to have the gauge glass full of solution of the same density and at the same temperature as that contained by the large tank. The other valve was at the end of the 1-in. rubber tubing, and served to cut off the flow from the tank when it was desired to empty the gauge glass. It is important to have the valve which empties the gauge glass immediately at the bottom of that glass, rather than next to the valve beside the tank. The object of this is to empty the gauge glass without completely emptying the hose, otherwise, when it is attempted to fill the gauge glass again, air is likely to be entrained in the solution and thus cause some difficulty in securing a perfect meniscus in the gauge glass.

A small 3-in. centrifugal pump, driven by a small motor, was used to circulate the water. It was connected, in about the manner shown diagrammatically in the sketch, to the dissolving box and salt-solution tank. Instead, however, of its suction dipping down into the head-race, as shown in the sketch, it was connected directly to the city water supply system. In using the city water supply, the pump tender must take great care that the full city pressure is not allowed to act on the pump casing, as it is likely to burst.

It will be seen from the arrangement of the 3-in. piping and valves that the circulating pump serves a double purpose. It is capable of circulating the water up through the dissolving box, over through the 8 by 10-in. trough and into and through the 5 000-gal. tank; or, it may simply circulate the solution in the tank by drawing it off through the suction and discharging it back into the bottom, the salt dissolving box under these circumstances being cut off by a valve in the 3-in. pipe. The object of this latter operation is to secure a perfect mixture in the salt-solution tank at a uniform temperature; also to dissolve

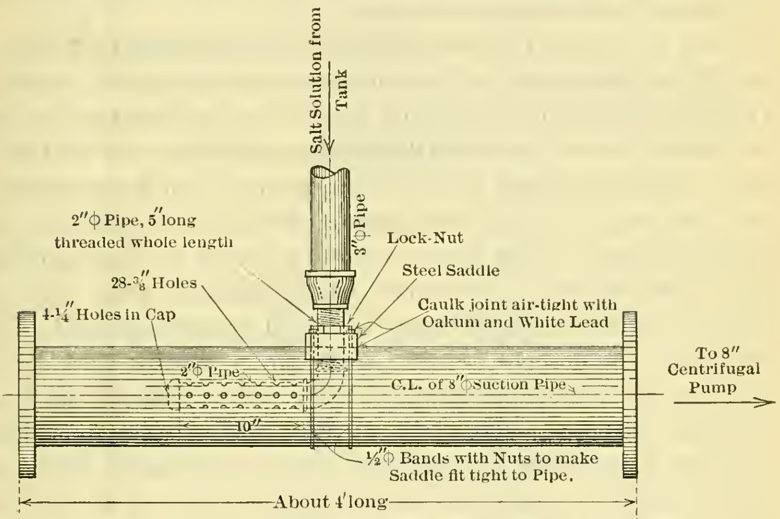
any salt which may find its way into the 5 000-gal. tank and settle to the bottom.

The operation of charging the tank with salt solution consists in, first, emptying and cleaning the large tank. This may be done through an opening in the bottom which is provided with a large nipple and gate-valve. The small circulating pump is then started, the valve leading to the dissolving box being open and the water being taken from the city supply, the valves in the 3-in. pipes leading to the large tank, of course, being closed. As soon as the water begins to flow into the dissolving box, salt is shoveled in at the top. The salt settles down through the water which is rising in the dissolving box and flowing over into the large tank through the 8 by 10-in. trough. The rate of water supply and the rate at which salt is shoveled into the top of the dissolving box must be properly adjusted by trial in order to secure the maximum efficiency. It was found that the capacity of the circulating pump was just about sufficient, when somewhat throttled down, to dissolve salt at the rate at which two men could shovel it in at the top. One or two attendants are required to keep the screens free from débris as the water flows into the solution tank. It was not found necessary to remove the hood during the operation of charging the tank, and this indeed would not be desirable at any time during the operation, as it would be likely to allow the admission of débris into the solution tank with the consequent danger of plugging the distribution system, especially at the point where the water is admitted into the suction of the 8-in. centrifugal pump which supplies the distributing sprinklers.

There was always a considerable percentage of fine sand carried by the salt, and this passed through all the screens and fell into the large tank, after which it settled on the bottom to a depth of $\frac{1}{8}$ to $\frac{1}{4}$ in. for each charge. The tank was not always cleaned after each filling, as this layer of sand caused no inconvenience. The sand was very fine and compact, and was not likely to be dislodged by currents of salt solution in the tank. The time required to charge the tank varied from somewhat less than 1 hour to 2 hours or more, according to circumstances.

A 3-in. pipe led from the bottom of the salt-solution tank to the suction of the 8-in. centrifugal pump, shown on a scow in Fig. 9. This scow was not used at any time in the tests. Originally, the object in

having the scow was to maintain a constant suction head on the large centrifugal pump, but it was not found necessary to have a uniform supply of water from this pump and consequently no scow was necessary. Had the scow been used, it would have been necessary to have flexible joints in the pipe lines leading to the centrifugal pump. As a matter of fact, the pump was on the deck of the power-house platform immediately over one of the piers of the head-race. In this position the suction lift varied from 17 to 23 ft., but the pump seemed to work very satisfactorily under any of these conditions of lift. There



DESIGN OF NOZZLE
FOR INJECTING SALT SOLUTION
INTO SUCTION OF 8-INCH CENTRIFUGAL PUMP.

Fig. 10.

was no screen over the intake of the 3-in. pipe leading to the 8-in. pump. The writer is inclined to believe that it would have been wiser to have had a screen over the intake of the 3-in. pipe, although there was no difficulty in this respect during any of the tests.

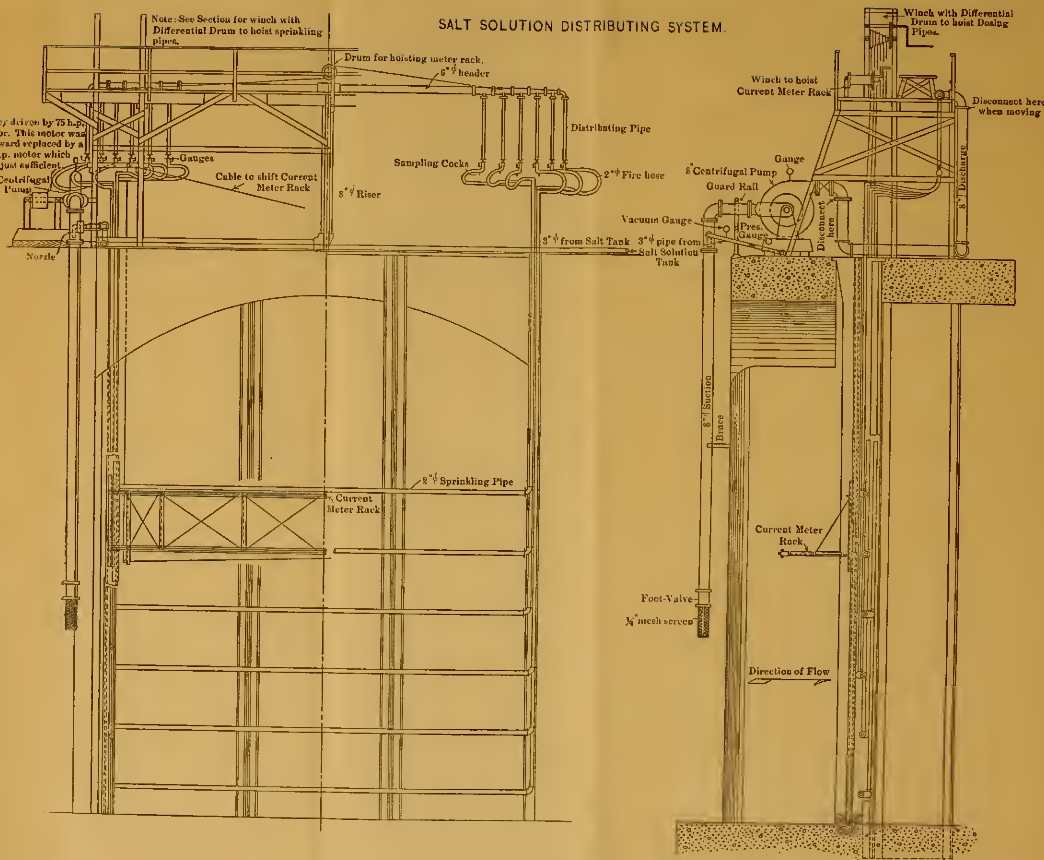
Fig. 10 shows diagrammatically the method of introducing the salt solution into the suction of the 8-in. centrifugal pump. The sprinkler attached to the elbow within the 8-in. suction points against the flow of the water on its way from the canal water level to the pump. This sprinkler was 10 in. long and had twenty-eight $\frac{3}{8}$-in. holes through its convex surfaces, and four $\frac{1}{4}$-in. holes in the cap.

It may not have been necessary to provide the end of the 3-in. salt-solution pipe with such a sprinkler, but the writer is of the opinion that this aids materially in securing a perfect mixture of salt solution with the feed-water for the centrifugal pump. There was little time for experimentation in this respect, so that there was no evidence whatever bearing on the design of such a sprinkler. It would appear to the writer that the holes for this sprinkler should have been smaller, so as to project jets of solution as far as possible into the body of the suction water feeding the pump. If the holes are too small they will plug when the solution carries solids.

Plate LIII shows transverse and longitudinal sections of the head-race at the point where the distributing sprinkler system was installed. The general method of distributing the salt solution, discharged by the 8-in. pump to the six horizontal distributing sprinklers in the head-race, was to lead the discharge of the 8-in. pump into a 6-in. header branching to the east and west, which discharged from each branch into six 2-in. pipes leading down into the stop-log slot at the corresponding side of the head-race. The twelve 2-in. distributing pipes were arranged so that they supplied the six horizontal sprinklers in pairs. The individual pipes of any one pair passed down on opposite sides of the head-race and were connected by elbows at the ends of the 2-in. sprinklers extending across the race. Each pair of risers from these distributing sprinklers was suspended by $\frac{1}{4}$-in. steel cables from two differential pulleys, one on the east and one on the west, arranged so that, by gears and cranks, the rotation of the pulleys would cause the pipes to rise and fall in such a manner that they were always equidistant, thereby dividing the cross-section of the head-race into six equal horizontal strips. The object of this was to keep the salt solution as uniformly distributed over the cross-section of the head-race as possible, and to permit of adjusting the levels of the pipes to the level of the surface of the water in the canal, which was likely to vary 6 or 8 ft. from time to time.

The necessity for this adjustment demanded flexible connections between the distributing pipes leading from the 6-in. header and the risers leading up from the sprinklers. This was accomplished by using sections of 2-in. hose of sufficient length to admit of the necessary motions. See Figs. 22 and 23.

SALT SOLUTION DISTRIBUTING SYSTEM.



The 6-in. header which has just been briefly described was carried across the top of a wooden platform resting on a frame, which is shown by Fig. 11. This will be better understood by examining Figs. 22 and 23, which are explained in detail in Section 97.

92.—Current-Meter Rack and Platform.—The platform, Fig. 11, which has just been mentioned, was constructed primarily for the purpose of suspending the current meters, which were carried on a steel frame or rack. This meter rack, Fig. 12, was constructed in two sections or leaves which were capable of folding so as to admit of the whole being dropped through the head-gate opening and into the stop-log slots, where it rested against two steel guides composed of two channels. These channels were provided at the bottom with a pair of legs, or spreaders, so that the channels could be dropped down the stop-log slots in a vertical position in such a manner that when the spreader came in contact with the bottom the legs would spread apart and hold the channel rigidly in its proper position at the bottom. The channels were then made fast at the top, so as to be practically in a vertical position, thereby acting as guides for the current-meter rack at each side of the head-race. The object of this arrangement was to admit of placing and removing the guides without the necessity of shutting down the unit or emptying the head-race. These channels seemed to work very satisfactorily, and could be placed and removed in a few minutes. The current-meter rack was suspended at each end by a $\frac{1}{4}$-in. steel cable which led vertically up to a pulley at the corresponding end of the current-meter platform; these two cables then passed around the drum of a small winch, by which the rack could be raised or lowered at pleasure.

In order to lower the rack into the head-race, its two leaves were closed and the whole was dropped through the head-gate opening into the stop-log slot until it was low enough to admit of opening the horizontal leaf into position. The current meters had been previously attached to the rack, so that they were then ready for operation. The rack with the current meters could thus be lowered to the bottom of the head-race or to any intermediate position desired. The horizontal leaf of the rack operated on vertical pivots, so that it could be shifted toward one side or the other of the race, as indicated by the dotted lines in Fig. 12, only one-half of the meter rack being shown. The rack was equipped with five pieces of angle iron, about 1 ft. $7\frac{1}{2}$ in.

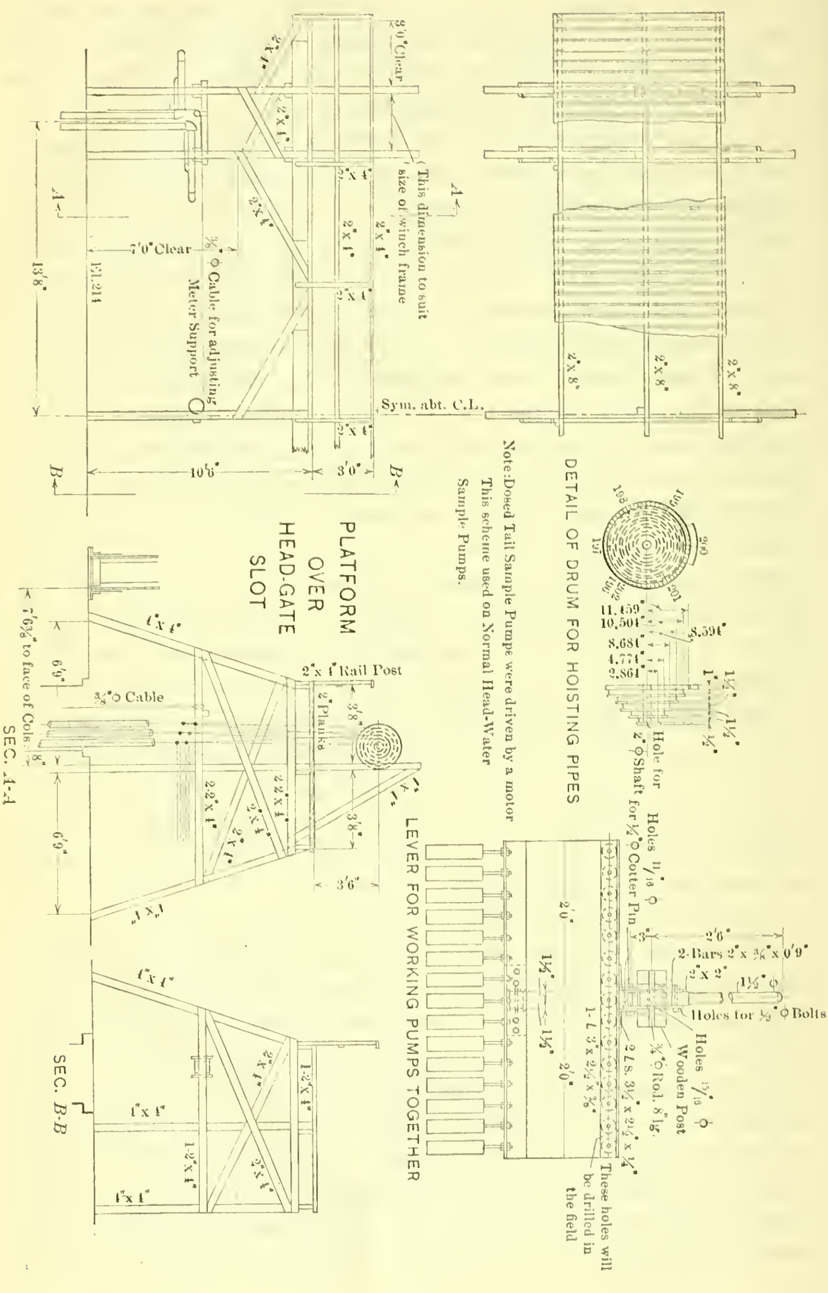
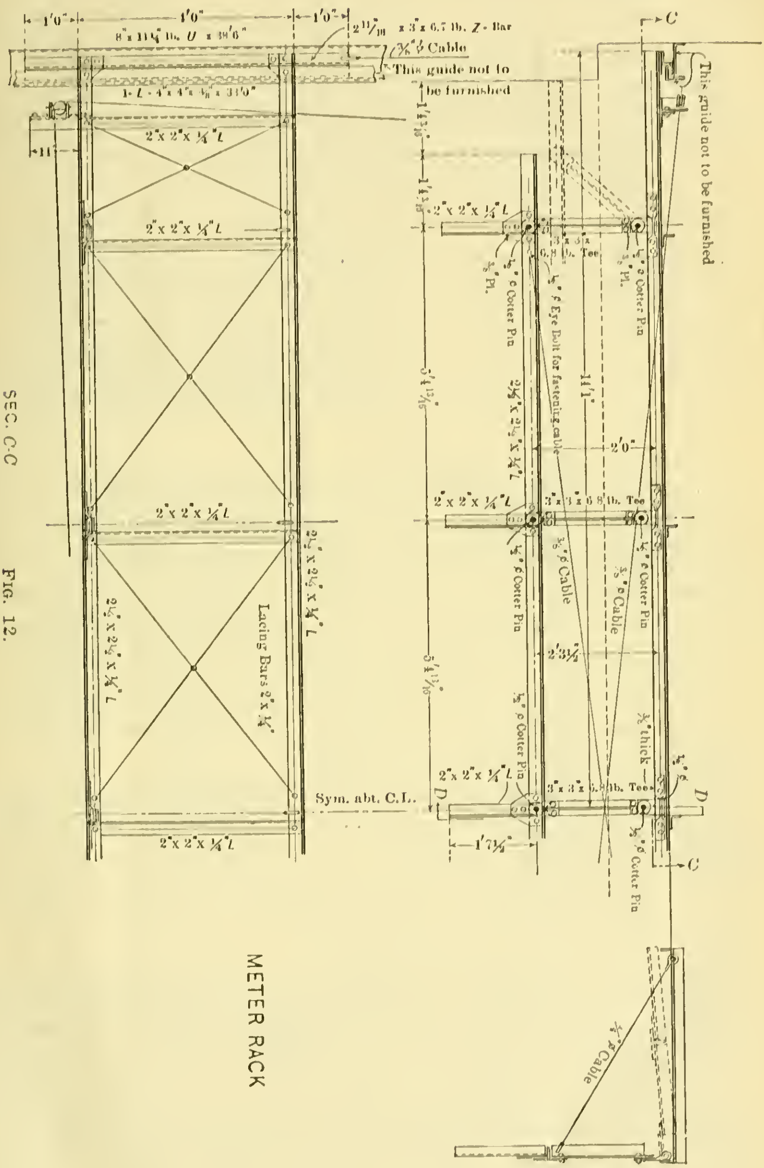

PLATFORM
OVER
HEAD-GATE
SLOT
This dimension to suit size of winch frame
3'0" Clear
7'0" Clear
3/8" φ Cable for adjusting Meter Support
El. 214
Sym. abt. C.L.
DETAIL OF DRUM FOR HOISTING PIPES
Note: Dosed Tail Sample Pumps were driven by a motor
This scheme used on Normal Head-Water
Sample Pumps.
2" φ Shaft for 1/2" φ Cotter Pin
Hole for
2-Bars 2" x 3/4" x 0'9"
Holes for 1/2" φ Bolts
Wooden Post
3/4" φ Rol. 8" lg.
2 Ls. 3 1/2" x 2 1/2" x 1/4"
1-L 3" x 2 1/2" x 3/8"
These holes will be drilled in the field
LEVER FOR WORKING PUMPS TOGETHER
2" x 4" Rail Post
2" Planks
3/4" φ Cable
7'6 3/8" to face of Cols.
SEC. A-A
SEC. B-B

FIG. 11.



SEC. C-C

FIG. 12.

METER RACK

SEC. D-D

long, and these were always parallel to the axis of the head-race, which was approximately parallel to the direction of the current. To these five pieces of angle iron the five current meters were rigidly attached, with their axes pointing directly up stream. There were two steel cables, arranged as shown in Fig. 12, leading around pulleys at each end of the rack, for the purpose of enabling the current-meter tenders to shift the position of the meters from one side to the other. This permitted of two points of operation for each meter, thereby giving ten meter points for any elevation of the rack. Five of these meter points were determined by the position of the horizontal leaf when it was shifted to the west and five when it was shifted to the east, in any one elevation. The westerly position was limited by the contact with the west wall of the west end of the long angle iron extending across the raceway. The easterly position was limited by the contact of the other end of the angle iron with the east wall.

The general method of operating this rack was to divide the depth of water into ten equal parts and then operate the rack successively at the middle of each of these parts, thus giving ten elevations for the meter rack and therefore one hundred meter readings for one sweep of the head-race by the meter rack. As in the case of the six horizontal distributing pipes, the ten meter elevations were always adjusted to the elevation of the water surface, so that the horizontal leaf of the meter rack represented the median line of one of the ten equal horizontal strips into which the race was divided.

Of the five current meters used, two were Haskell meters of the screw type; two were Ott meters of the screw type; and one was a small Price meter, very kindly lent by H. W. King, M. Am. Soc. C. E., of the University of Michigan. The two Ott meters were lent through the courtesy of the Allis-Chalmers Company.

When it was desired to move the testing equipment from one unit to another, the meter rack was raised to its highest position under the platform; the connection of the discharge from the 6-in. header and the flexible hose connections to the risers from the sprinklers were removed; the connections of the cables to the risers from the sprinklers were also removed, the risers and sprinklers being temporarily lashed to a long I-beam. Then the entire platform with its equipment was shifted to one side, using the traveling crane, while the I-beam carrying the sprinklers and current-meter guides was being

moved by the crane for the purpose of lowering its load into the stop-log slots of the unit to be tested.

The suspension platform was then taken up by the crane and placed over the head-race into which the sprinklers and guides had been lowered, after which all its connections were put in place and the testing equipment was adjusted for use in its new location.

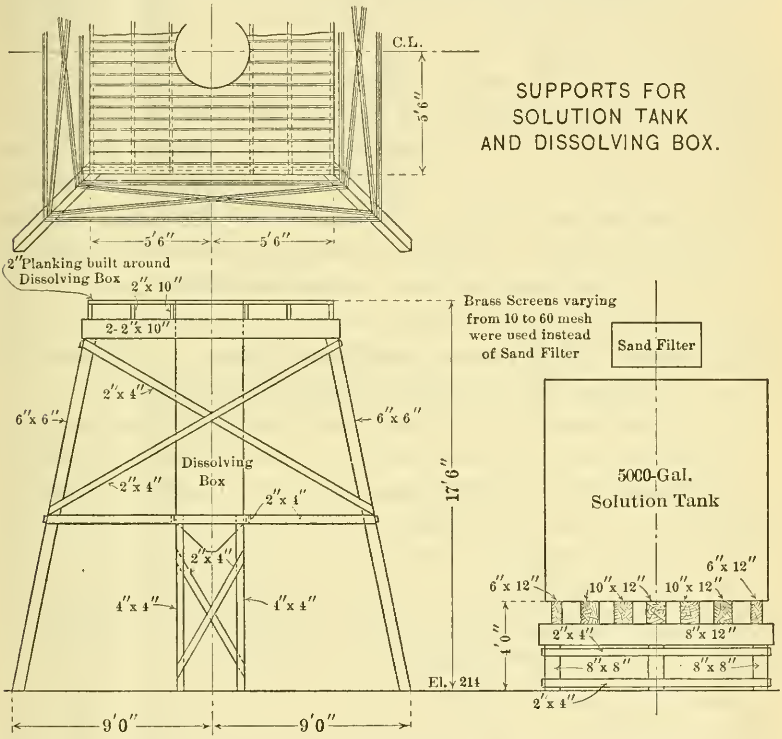


FIG. 13.

93.—Salt-Dissolving Box and Platform.—Fig. 13 shows the salt-dissolving box and platform. This box consisted of a white pine tank, 3 ft. in diameter and 10 ft. high. The staves were of 2-in. plank dressed to about $1\frac{7}{8}$ in. in thickness. The floor of the platform was constructed so that the top of the tank projected about 10 or 12 in. above it. At the edges of the floor there was a 2-in. plank bulwark, about 18 in. high, so that salt could be thrown on the platform without rolling off. In constructing another such platform, the writer would place the dissolving box eccentrically, relative to the platform,

as it is not necessary for the men to stand on opposite sides of the box. With such a scheme, the size of the platform might be made somewhat smaller, and at the same time more convenient. In making use of this box, an ordinary sand screen, of about $\frac{1}{4}$-in. mesh, is placed over its top opening in such a manner that when salt is thrown on the screen it will fall through and into the box. The trough leading to the salt-solution tank takes its solution from the top of the box through a notch, 8 in. deep and 10 in. wide. Care must be taken to screen this notch from the salt which is falling into the box so as to prevent particles from being carried over into the salt-solution tank. It is probable that some salt is carried over in this manner, but, if the quantity is limited, it is certain that it will soon be dissolved—probably before it reaches the bottom of the large tank. The screen consists of a vertical diaphragm dividing the horizonal section of the box into two segments; the larger of these receives the falling salt, and the smaller acts as an opening through which the salt solution is supplied to the trough. This diaphragm should extend down into the box 2 or 3 ft. so as to afford better protection to the notch at the top.

94.—Perforations in Sprinkling Pipes.—Fig. 3, showing the design of the sprinkler holes, has already been described in Sections 56 and 57. It will be seen that the perforations consist of a number of groups of $\frac{5}{32}$ and $\frac{7}{64}$-in. holes. The diameters of these holes are not exactly of the dimensions intended, though they serve fairly well the purpose for which they were made. A group consists of two of the larger and six of the smaller size, as shown in Fig. 3. The object of these holes, of course, was to spray the salt solution as uniformly as possible into the cross-section of the head-race.

Some difficulty was caused by scale becoming loosened from the interior surface of the pipe system and plugging the holes. It would seem desirable, therefore, to have the holes sufficiently large to eliminate this trouble. Holes $\frac{1}{4}$ in. in diameter would serve this purpose well, but in any case it would be advisable to have a T on each sprinkler pipe with a perfectly tight valve on the extremity, provision being made for the operation of the valve from above at such times as it might be desired to flush out the pipe system. This was not done in the tests described, but it is suggested as a safeguard in other tests. No serious difficulty, however, was caused by the plugging of the holes after the pipes had been thoroughly flushed once or twice

and blown out by compressed air. It must be carefully borne in mind, in designing sprinklers, that the centrifugal pump should be properly proportioned to project the jets far away from the orifices, for the purpose of reaching all parts of the cross-section with salt solution. It would be preferable, therefore, to have fewer holes, thus creating a larger head for the injection of salt solution, rather than to have the holes too numerous and thus lose the opportunity for a good distribution.

95.—Tail-Race Sampling System.—The tail-race sampling pipes were first installed in the stop-log slot at the mouth of the draft-tubes or tail-race. These pipes consisted of six horizontal ¼-in. pipes, uniformly spaced upward and downward over the cross-section of the tail-race, as shown on Plate LIV. These ¼-in. pipes had ⅛-in. perforations, uniformly spaced about 6 in. apart along the up-stream face of the pipe. The whole system of pipes was carried in a frame of light angle iron, and, when disconnected from the pipes, connecting the sampling pipes with the sampling pumps in the chamber immediately above the upper draft-tube, the whole could be lifted out of the tail-race by a hand-crane. This arrangement was not very satisfactory, owing to the frail nature of this light pipe and construction in general. The pipes broke at the joints, and caused considerable trouble in this way by reason of leaks. Had the fittings been of malleable iron, as was specified, there probably would have been no such trouble.

The pipes were plugged at their centers, so that there were in reality twelve sampling pipes, six in the west and six in the east half of the tail-race. These were numbered from 1 to 12, the odd numbers being on the west and the even numbers on the east, progressing downward.

It gradually became evident that the sampling pipe system could be improved by moving the whole back to the plane of the 4-in. pipes, which had been provided during the construction of the power-house for the admission of Pitot tubes into the tail-race at the mouths of the draft-tubes. This plane is about 8 ft. farther up stream, and has the advantage that the suction pipes leading to the sampling pumps do not pass through a layer of water which was not discharged from the turbines being tested, as was the case at the lower section, where the risers from the sampling pipes passed up through 6 or 8 ft. of

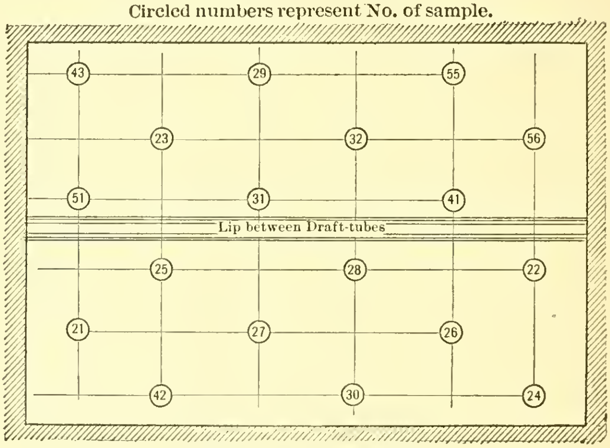
water which was swept back against the power-house wall and probably contained little or no salt other than that naturally in the river.

The caisson compartment shown in the lower part of Plate LIV was merely a suggestion as to how the samples might be taken by gravity without the necessity for sampling pumps. This caisson was neither constructed nor used in the tests.

The battery of eighteen sampling pumps stood in the chamber above the upper draft-tube, and was driven by a 10-h.p. motor. It will be described more completely in Section 97.

SKETCH SHOWING LOCATION OF SAMPLES TAKEN IN TAIL-RACE ON UPPER SECTION.

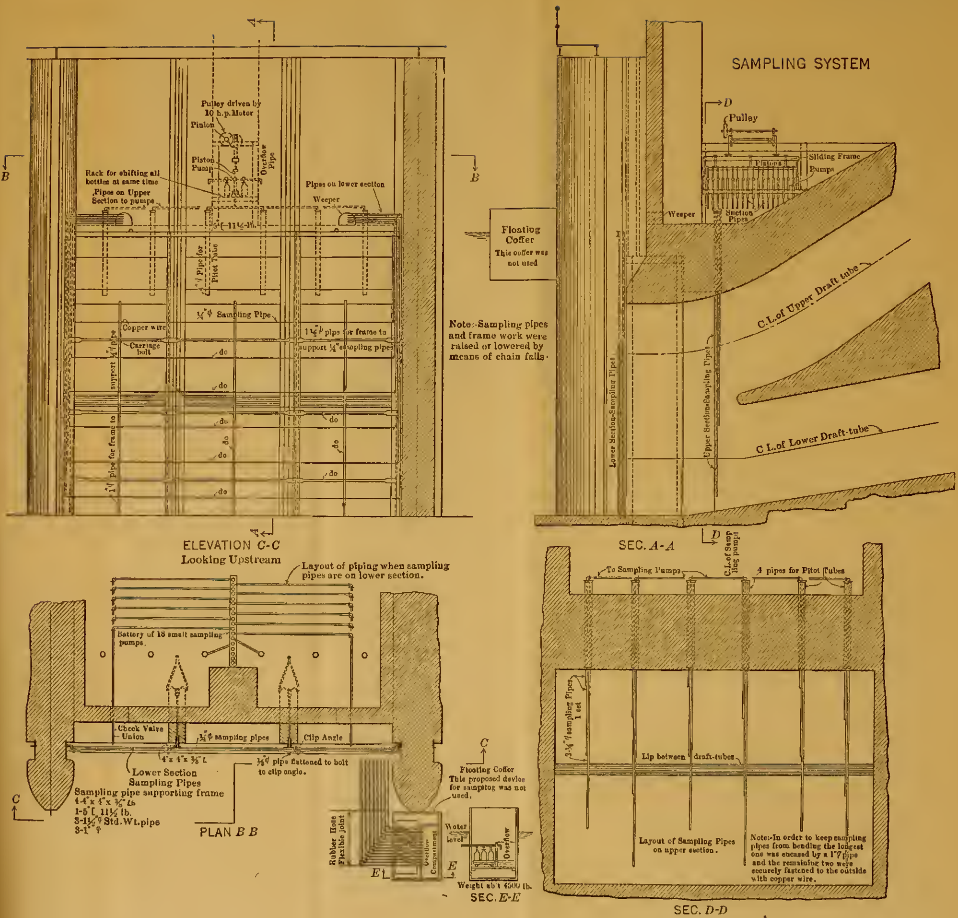
Circled numbers represent No. of sample.



SECTION OF DRAFT-TUBE LOOKING UP STREAM.

FIG. 14.

Fig. 14 shows the relative locations, in the upper sampling section, of the intakes of the eighteen sampling pipes which extended down through the 4-in. Pitot tube admission inlets. The numbers of the samples were the same as those shown in Fig. 14. It appears that there was no particular system in connecting up the sampling pipes with the sampling pumps, so that the numbers are not arranged as systematically as they are in the case of the sampling system at the lower section. However, the sample bottles were carefully numbered so as to correspond to the particular locations to which the same numbers are attached in the illustration, so that no ambiguity could result as


SAMPLING SYSTEM
Pulley driven by 10 h.p. Motor
Pinion
Piston Pump
Overflow Pipe
Rack for shifting all bottles at same time
Pipes on Upper Section to pumps
Pipes on lower section
Weeper
Pipe for Pitot Tube
¼"φ Sampling Pipe
Copper wire
Carriage bolt
1¼"φ pipe for frame to support ¼" sampling pipes
do
1¼"φ pipe for frame to support
ELEVATION C-C
Looking Upstream
Floating Coffer
This coffer was not used
Note:-Sampling pipes and frame work were raised or lowered by means of chain falls.
Pulley
Sliding Frame
Pumps
Suction Pipes
Weeper
C.L.of Upper Draft tube
C.L.of Lower Draft-tube
Lower Section-Sampling Pipes
Upper Section-Sampling Pipes
SEC. A-A
Layout of piping when sampling pipes are on lower section.
Battery of 18 small sampling pumps
Check Valve
Union
¼"φ sampling pipes
Clip Angle
4"x 4"x ⅜" L
¼"φ pipe flattened to bolt to clip angle.
Lower Section
Sampling Pipes
Sampling pipe supporting frame
4-4"x 4"x ⅜" Ls
1-5" [11½ lb.
3-1½"φ Std.Wt.pipe
3-1"φ
PLAN B B
Floating Coffer
This proposed device for sampling was not used.
Rubber Hose Flexible joint
Overflow Compartment
Water level
Overflow
Weight ab't 4500 lb.
SEC. E-E
To Sampling Pumps
C.L.of Sampling pumps
4 pipes for Pitot Tubes
3-¼"φ sampling Pipes 1 set
Lip between draft-tubes
Layout of Sampling Pipes on upper section.
Note:-In order to keep sampling pipes from bending the longest one was encased by a 1"φ pipe and the remaining two were securely fastened to the outside with copper wire.
SEC. D-D

to the location in the tail-race from which a particular sample was taken.

96.—Relation of Testing Plant to Turbines.—Plate LV shows sections of the water passages for the turbine unit being tested, and also the head-race and penstock in which the turbines were placed. The upper and lower sections for the tail-race sampling system are also indicated in the longitudinal section. The relative location of the dissolving box and solution tank is also shown.

97.—Photographs of the Testing Equipment.—Fig. 15 is a general view showing the arrangement of salt-solution tanks, salt distributing system, and current-meter suspension frame.

The salt was brought to the plant by rail, in carloads of 20 to 30 tons, and stored in bins at the west end of the gate-house beyond the extreme right of Fig. 15. The total capacity of the bins was about 100 tons, but probably they did not contain more than 80 or 90 tons at any one time. The hatchways of the bins were immediately under the traveling crane, so that the salt was readily handled by this crane in connection with a second crane, at the junction of the gate-houses of the old and new power-houses.

The salt was carried by the crane in a large skip, having a capacity of slightly more than 1 ton. This was lifted to the platform of the dissolving box, where it was dumped. Two men then shoveled the salt rapidly into the box while the circulating pump took water from the city main, pumped it up through the dissolving box and over into the salt-solution tank through the 8 by 10-in. trough previously described.

A 3-in. supply pipe led the salt solution from the tank to the suction of the 8-in. centrifugal pump, at the left center of Fig. 15, immediately under the shed.

A section of the 3-in. pipe can be seen at the point where it enters the 8-in. suction of the centrifugal pump. The back of the pressure gauge on the 3-in. pipe is also visible. The centrifugal pump can be seen, and also the wooden belt-guard leading from the pump to the motor.

The elevation of the water surface in the canal was evidently about 195.7 ft., so that the suction lift of the centrifugal pump was about 21.3 ft., the center of the pump being approximately at Elevation 217.

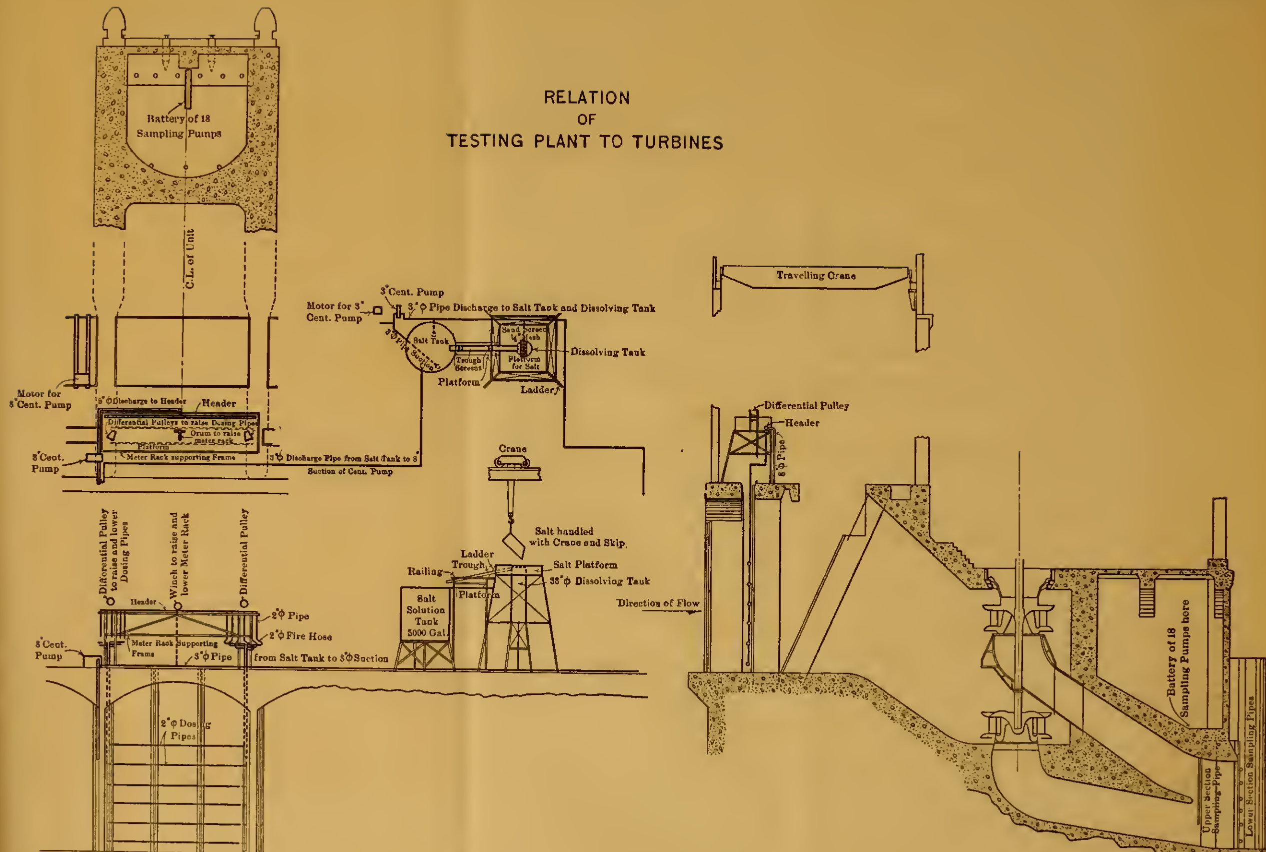
The salt-solution gauge reader can be seen on the gauge reading scaffold at the side of the gauge-board attached to the salt-solution tank. Two of the head-water sampling pumps, with their suction pipes leading down into the canal, are visible at the center of Fig. 15. Several of the 1-gal. sampling bottles are also visible beside the sampling pumps. One of the gauge-boards can be seen attached to the concrete wall of the entrance to the head-race of the turbine being tested. The head-water was not read at this cross-section, but at a point 5 ft. down stream from the trash racks, which are not visible in Fig. 15. The gauge-board indicates an elevation of water surface of about 195.7. There was another gauge on the opposite wall, and the two were used for the purpose of giving the elevation of the canal water at the current-meter section on each side.

The current-meter suspension frame and platform is at the left center, back of the centrifugal pump and shed. The 8-in. riser supplying the header on top of the platform, and also the top of the header itself, can be seen at the back part of the framework. The two winches carrying the differential pulleys at the ends of the platform, as well as the winch for the current-meter rack in the center of the platform, can be seen.

Fig. 16 shows the salt dissolving box and salt-solution tank. The iron pipette for calibrating the tank stands between the tank and the right-hand leg of the frame supporting the salt platform. It consisted of a length of 12-in. pipe held in a four-legged wooden frame. The 2-in. pipe carrying the water glass was not attached to the top of the iron pipette at the time of taking the photograph. The 10-ft. gauge-board carrying the 1-in. glass tube in a slot along its center, together with the rubber tube attaching the bottom of the glass gauge to the bottom of the tank, is at the right of the salt-solution tank. The gauge reader's scaffold stands at the right of the gauge-board. The 3-in. riser leading from the circulating pump to the bottom of the salt dissolving box is shown, also the pipe leading under the planks from the city water supply at the left. This pipe joins the pipe leading to the bottom of the salt dissolving box, and may be disconnected from it by a gate-valve. A portion of the crane which assists in handling salt is at the extreme right.

Fig. 17 illustrates the operation of charging the salt-solution tank. A skip-load of salt has just been raised into position by the crane and

RELATION OF TESTING PLANT TO TURBINES



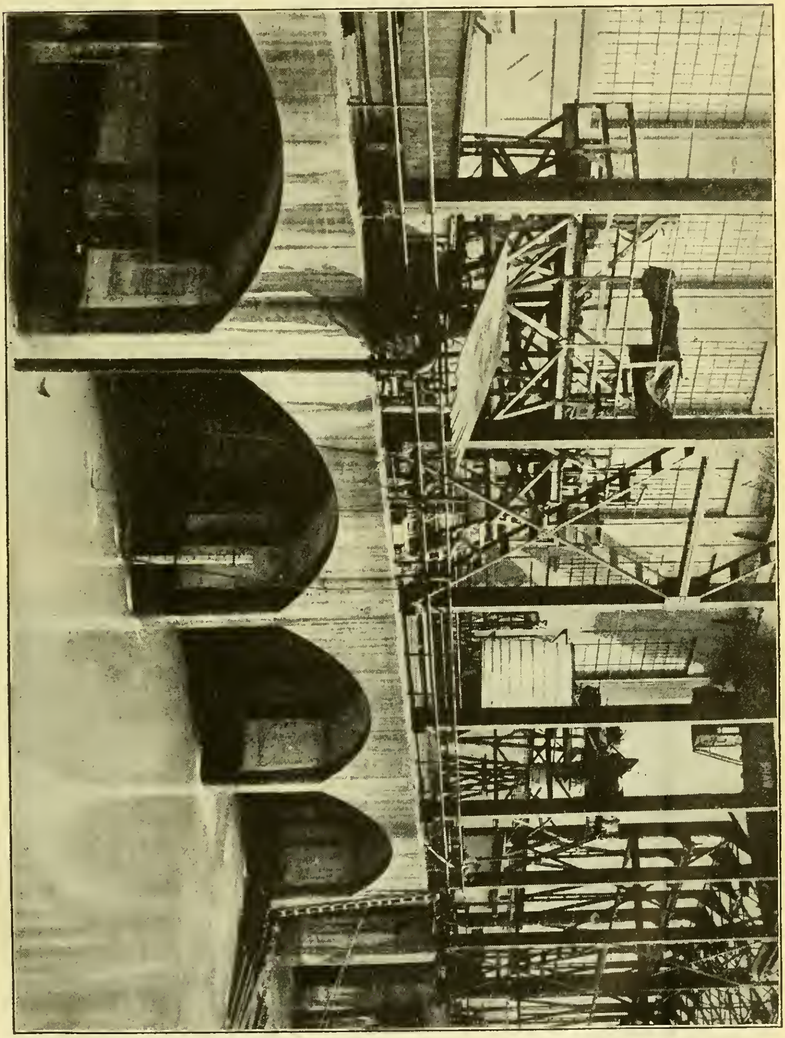


FIG. 15.—SALT-SOLUTION SYSTEM AND CURRENT-METER SUSPENSION FRAME FROM EAST BANK OF CANAL.

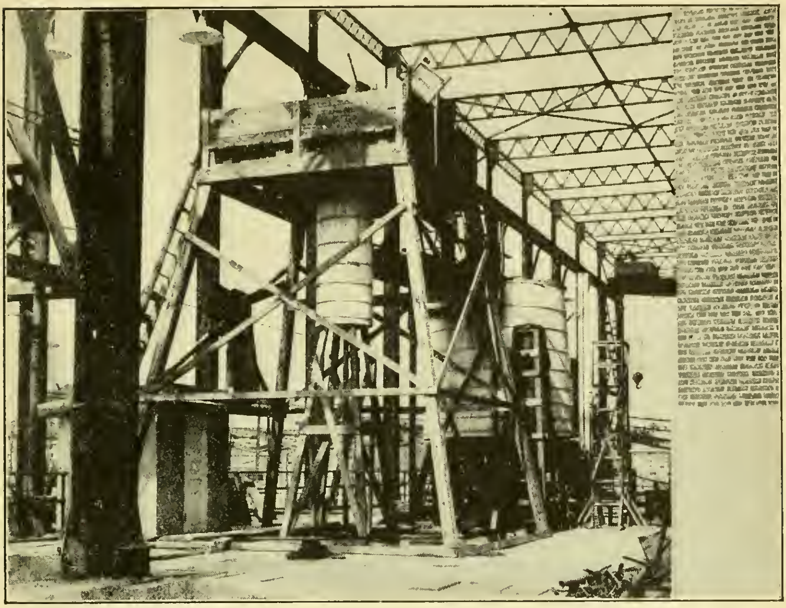


FIG. 16.—SALT-DISSOLVING BOX AND SALT-SOLUTION TANK.

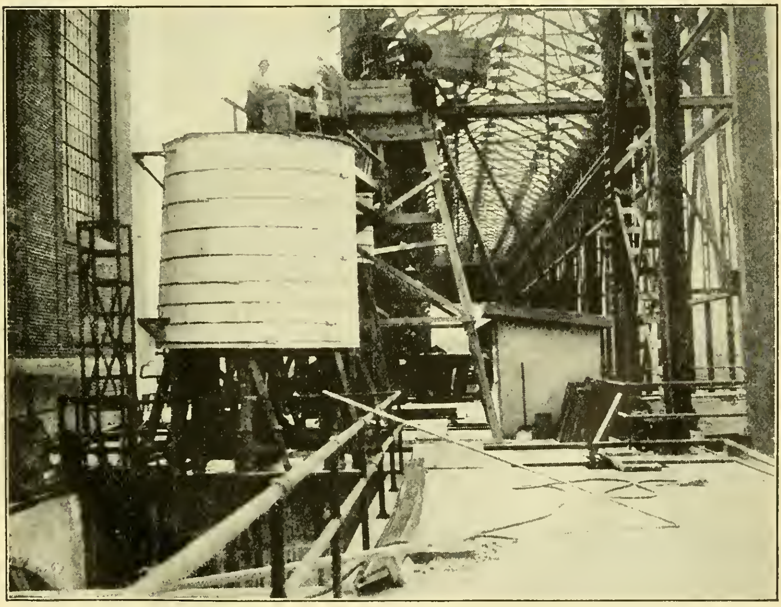


FIG. 17.—CHARGING SALT-SOLUTION TANK.

is about ready to be dumped on the salt platform. The sand screen immediately above the upper opening of the salt dissolving box is in view.

One of the engineers may be seen knocking the débris from one of the screens which he has withdrawn from the slots in the 8 by 10-in. trough leading from the salt dissolving box to the hood immediately over the salt-solution tank. This hood can be seen at the end of the trough. The tank was provided with a cover to protect the surface of the water from any falling débris and also to keep off the wind. Very little trouble was caused by waves forming in the tank, but the writer would suggest having a grilled float on the surface of the water in such a tank, in order to still the surface as completely as possible. This was not done during the tests described herein, and the result was that on several occasions the meniscus of the salt-solution gauge was not as steady as it might have been, thus introducing small errors into the calculation of the quantity of salt solution consumed during the particular tests in question.

At the extreme lower left is seen the box in which was placed the motor driving the small circulating pump, which latter is in view but somewhat obscured by the multiplicity of pipes, timbers, cables, and chain in its vicinity. The discharge pipe can be plainly seen leading vertically upward from the discharge of the pump.

The 3-in. supply pipe leading from the bottom of the salt-solution tank to the suction of the 8-in. centrifugal pump can be seen leading across the floor of the gate-house and making a right angled turn near the lower right corner.

Fig. 17 also gives a perspective down through the west gate-house toward the salt storage bins and the chemical laboratory which is obscured by the small concrete house enclosing the motors for the ice sluice-gates, the gears for which are shown near the lower right corner.

Fig. 18 gives another view of the salt-solution tank and salt dissolving box connected by the 8 by 10-in. trough and hood.

The gauge reader may be seen at the left of the tank, making a record of the gauge reading.

The centrifugal circulating pump is much more plainly visible in this than in Fig. 17.

Fig. 19 gives a bird's-eye view of the salt dissolving box, salt-solution tank and deck of the current meter and sprinkling pipe suspen-

sion platform. The differential pulleys and twelve suspension cables which were attached to the risers from the six sprinkling pipes are at the ends of the platform, and the winch for raising and lowering the meter rack is at the middle of the up-stream side of the platform. The 6-in. header fed by the 8-in. riser from the centrifugal pump may be seen on the deck of the frame. The 8-in. centrifugal pump is under the shed at the extreme lower right of Fig. 19. The left center foreground shows the tops of the racks in one of the units tested.

The screen for salt and a quantity of salt piled on the salt platform are in view.

Fig. 20 shows the centrifugal pump driven by a 75-h.p. motor, and the 8-in. discharge leading to the 8-in. riser and header on the platform. Parts of the twelve 2-in. distributing pipes leading from the header, six at each end, may be seen. The lever for operating the four head-water sampling pumps, the cross-head connecting the four plungers, the four plungers themselves, and the bottom parts of the suctions of these small pumps can be seen just above a collapsed section of 2-in. hose pipe leading from one of the distributing pipes to the sprinkler system. These sampling pumps were small hand-pumps supplied by the Deming Company, and the handles can be seen just above the cross-head and under the hand-lever.

Fig. 21 shows the iron pipette calibrator used in the calibration of the large salt-solution tank. The top and bottom of this calibrator are not plainly visible, but they were each conical in shape. The cone at the top was for the purpose of allowing the escape of air through the 2-in. pipe at the top when the calibrator was filled with water; the conical bottom was to facilitate draining the calibrator when it was emptied through the throttle valve at the vertex of the cone. The water glass at the side of the 2-in. pipe projecting above the vertex of the upper cone was for the purpose of ascertaining the level of the water in the calibrator. A paper label can be seen pasted on this gauge. The label carried a mark at the proper elevation for a given capacity. The volume of water discharged from the calibrator was about 240 liters (8.466 cu. ft.) when it had been previously filled up to the mark with canal water.

The method of calibrating the tank was to set this calibrator down on the top of the rack platform, shown in Fig. 19, so that the water in

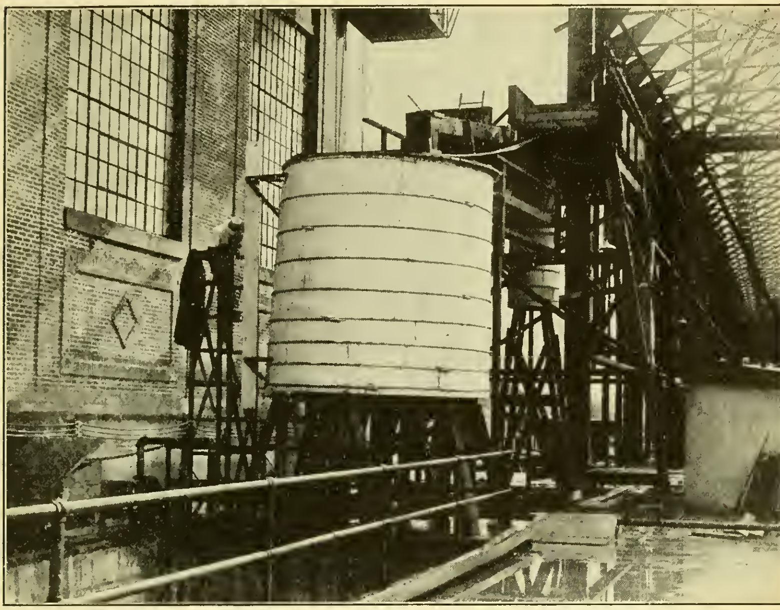


FIG. 18.—SALT-SOLUTION TANK, AND OBSERVER RECORDING READINGS OF SALT-SOLUTION GAUGE.

FIG. 19.—BIRD'S-EYE VIEW OF SALT-SOLUTION TANK AND DISSOLVING BOX.

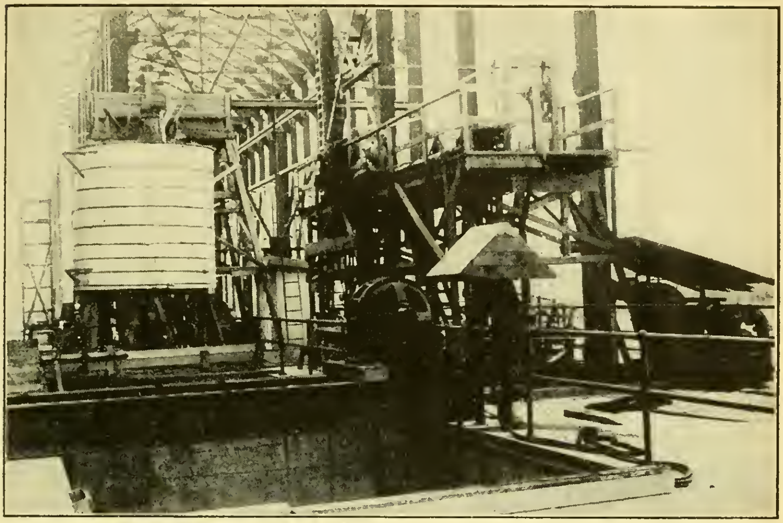


FIG. 20.—CENTRIFUGAL PUMP DRIVEN BY 75-H. P. MOTOR, AND 8-IN. DISCHARGE LEADING TO 8-IN. RISER AND HEADER ON PLATFORM.

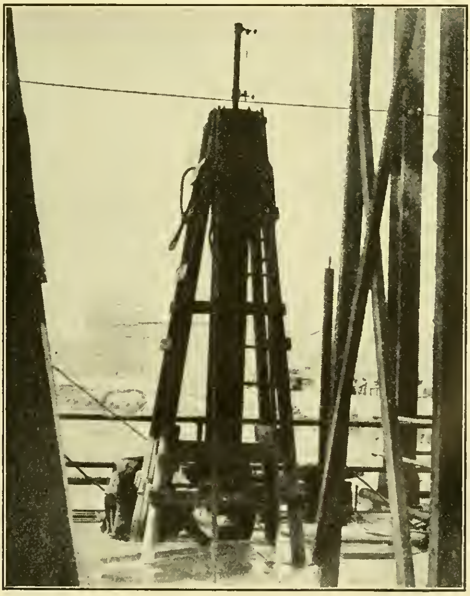


FIG. 21.—LARGE IRON CALIBRATOR. THE CONICAL TOP AND BOTTOM ARE NOT VISIBLE.

the salt-solution tank could be drawn by gravity through a hose into the 2-in. pipe on top of the calibrator.

The meniscus in the glass gauge was brought to the desired zero point on the scale by filling the tank up to the necessary level. A horizontal lead pencil mark was then made at the level of the meniscus on the right face of the gauge-board, beginning at the edge of the slot next to the glass tube and extending toward the outer edge of the board. This mark was the zero graduation. The calibrator was then filled by drawing water into it from the tank. After adjusting the level of the water in the calibrator very carefully by allowing the water to run slowly from the tank into the calibrator, when nearly full, another mark was made at the lower level of the meniscus in the glass tube. The calibrator was then emptied by opening the throttle valve at the bottom, the water supply from the tank, of course, being cut off. After this the process was repeated until the gauge-board had been calibrated down to the desired limit.

The throttle valve at the bottom of the calibrator had a small leak which was taken care of by slipping a bottle under the calibrator so as to capture the water which escaped by drops. By pouring this captured water back into the calibrator at the proper moment the error was eliminated.

This large calibrator or pipette, as it might be called, possessed the characteristic of an ordinary chemical glass pipette, in that its delivery differed from its contents. When the calibrator is discharged by drawing the water out of the bottom, the sides and angles of the calibrator retain considerable water by adhesion and capillarity. This difference was not determined by any decisive experiments during the tests, but it may be computed indirectly from the results of the gravimeteric and chemical determinations of its capacity.

Figs. 22 and 23 are very similar. They give fairly good views of the current-meter suspension frame and platform. The differential pulleys and cables suspending the sprinkler system may be seen, and also the risers leading from the 6-in. header on the deck over the framework. At the left center one may see the gate-valves on the risers, the pressure gauges and elbows connecting by means of nipples with the 2-in. hose pipes which form the flexible joint between the rigid header and the adjustable sprinkler system. On the bottoms of the nipples may be seen the small pet-cocks from which the salt-solution

samples were taken when it was desired to ascertain how uniformly the salt solution was being distributed by the 8-in. centrifugal pump. There were twelve of these sampling cocks, and, as has been previously stated, the samples taken therefrom were absolutely uniform in all tests made for the purpose of studying the distribution of salt solution among the twelve risers.

The table on which the five current-meter observers made their records is in place, and the telephone, enabling communication with all the other observers connected with the test, is at the lower right center just above the right end of the table.

Figs. 22 and 23 give a better view of the layout of the 8-in. discharge, 8-in. riser, and 6-in. header than any of the other illustrations.

Fig. 24 shows the discharge from the uppermost sprinkler when slightly raised above the surface of the water in the head-race. It will be seen that there is a system of holes discharging vertically upward, another discharging horizontally up stream and a third discharging vertically downward. Each of the six sprinkler pipes was perforated in this manner, so as to give as perfect a mixture in the head-race as possible.

The scaffold, on which one of the engineers may be seen, was arranged for the purpose of facilitating the current-meter work. It was found more convenient to attach and displace the meters when working on this platform than to attempt to raise and lower the meter rack through the gate-slot in the floor of the gate-house. The current-meter rack may be seen immediately back of the platform.

Fig. 25 shows the current-meter rack carrying the five current meters, as in a regular test. The rack is raised sufficiently to elevate the meters above the water surface.

Fig. 26 shows the battery of eighteen tail-race sampling pumps connected to the frame to which was attached the gear driving the cross-head which actuated the plungers of the pumps. The gears were driven by a 5-h.p. motor bolted to the pulley shown at the top. Nine of the pumps discharged toward the observer and nine in the opposite direction, so that nine bottles were placed on one side of the pumps and nine on the other. These sample bottles were placed on a board provided with cleats to prevent the bottles from slipping. One of these boards can be seen in the lower foreground connected with two levers. This device was used to shift the bottles simultaneously from under

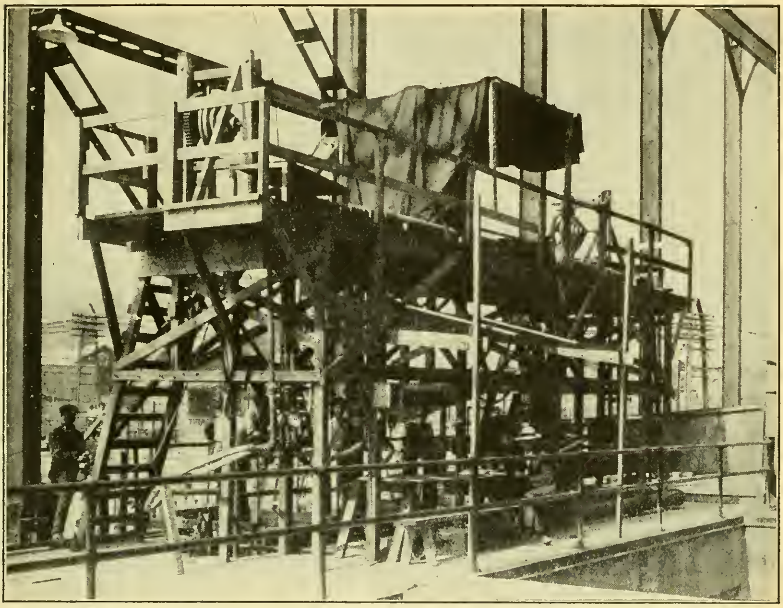


FIG. 22.—CURRENT-METER SUSPENSION FRAME AND PLATFORM, WHICH ALSO CARRIES THE DISTRIBUTING-PIPE SUSPENSION SYSTEM AND THE 8-IN. HEADER.

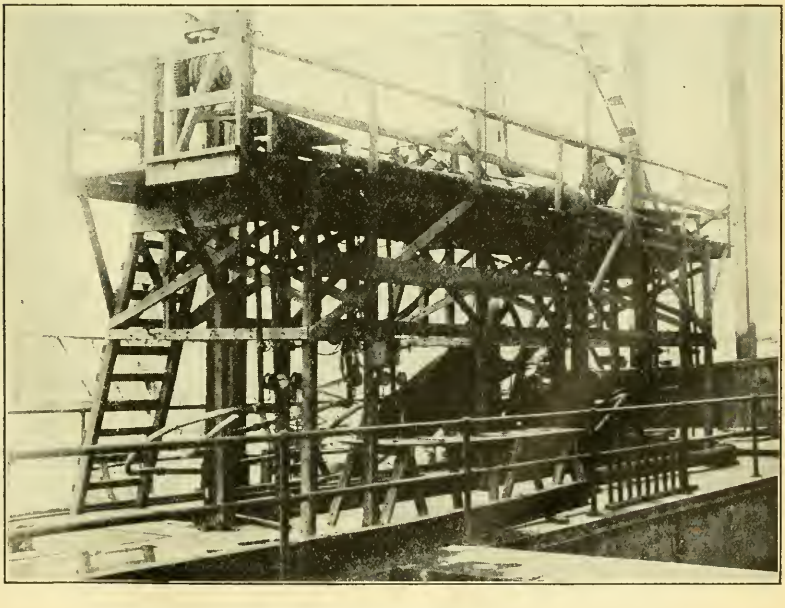


FIG. 23.—CURRENT-METER SUSPENSION FRAME AND PLATFORM.

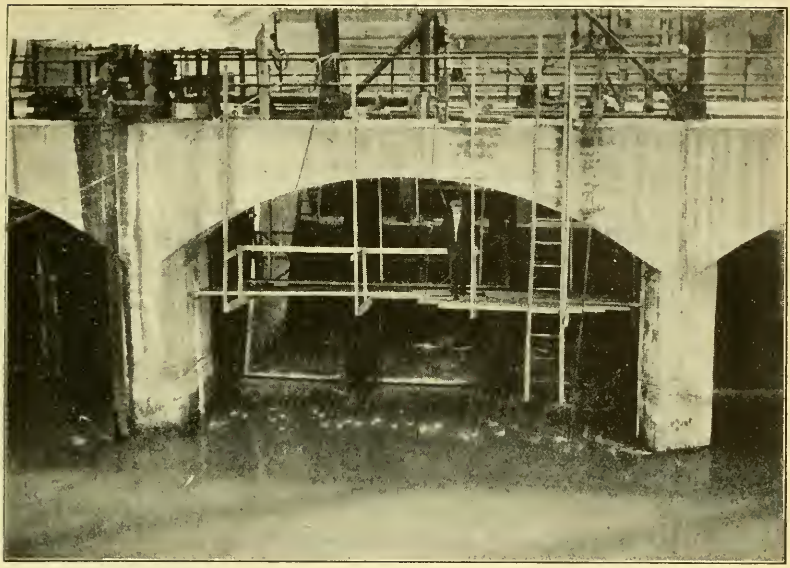


FIG. 24.—DISCHARGE OF UPPERMOST SPRINKLER OF SALT DISTRIBUTING SYSTEM.

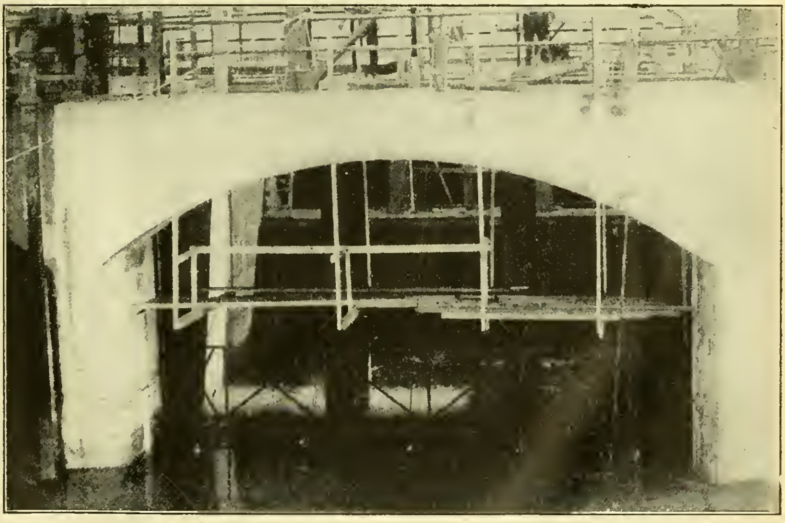


FIG. 25.—CURRENT-METER RACK, CARRYING FIVE METERS READY FOR TEST.

the discharges of the pumps when it was desired to stop the sampling. The levers were shifted in the opposite direction when it was desired to start the sampling. One attendant on each side of the battery of pumps took care of the sampling and watched the filling of bottles and the operation of pumps, so as to correct any interference with the proper taking of samples due to the failure of a pump or overflowing of a bottle.

Prior to the time of starting the samples, the attendants gauged each of the pumps, so that each bottle would be three-fourths full by the end of the test. The bottles were of the ordinary 1-gal. spring water type.

Fig. 27 shows the battery of separatory funnels and water-baths carrying the casseroles into which the separatory funnels discharge when the attendant opens the valve slightly so as to admit a small quantity of sample into a particular casserole.

The water-baths were heated by especially constructed electric heaters, the electrical switches for which may be seen on the wall back of the water-baths.

Fig. 28 shows one of the asbestos board trays made to support a 14 by 28-in. water-bath over the coil of nichrome resistance wire shown in the bottom of the tray. These heaters were very satisfactory, and the nichrome wire is very durable, even at high heats. The writer is indebted to Mr. G. K. Herzog for this design.

Fig. 29 is a view in the laboratory, showing the sample-storage shelves on the left and the bottle-draining racks on the right. The draining rack on top of the storage compartments on the right has no bottles on it. It was made for 1-qt. cream bottles. The larger lower rack carries 1-gal. bottles, a few of which are in place. Two of the 1-gal. sample-bottle carriers and one of the 1-qt. bottle sample carriers are shown on the floor below the draining rack.

Fig. 30 shows the large shunt used in testing the direct-current units. The shunt is in place as it was attached to Unit No. 12. The basin in the center of the shunt contains kerosene into which a thermometer hangs for the purpose of furnishing the temperature of the shunt when desired.

This shunt is of peculiar construction, and was designed by Mr. I. B. Smith, of the Leeds and Northrup Company, Philadelphia, Pa.

Fig. 31 shows the voltmeter and ammeter potentiometers which

were used in determining the direct-current electrical power across the shunt, shown in Fig. 30, and between the terminals of the generator. These instruments were also manufactured by the Leeds and Northrup Company. They are extremely sensitive, and the direct-current power readings are very accurate, probably being within 0.1% of the truth. The instruments are even more accurate than this under laboratory conditions.

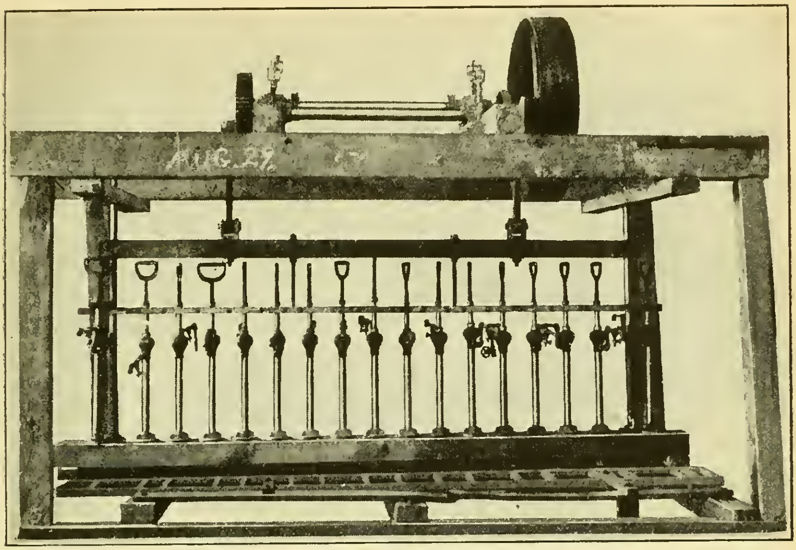


FIG. 26.—BATTERY OF EIGHTEEN TAIL-RACE SAMPLING PUMPS.

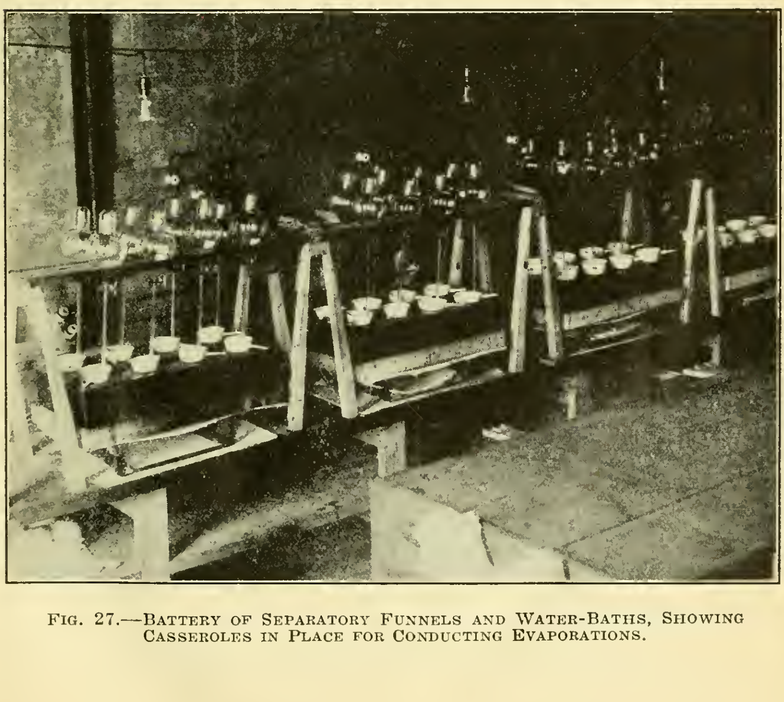


FIG. 27.—BATTERY OF SEPARATORY FUNNELS AND WATER-BATHS, SHOWING CASSEROLES IN PLACE FOR CONDUCTING EVAPORATIONS.

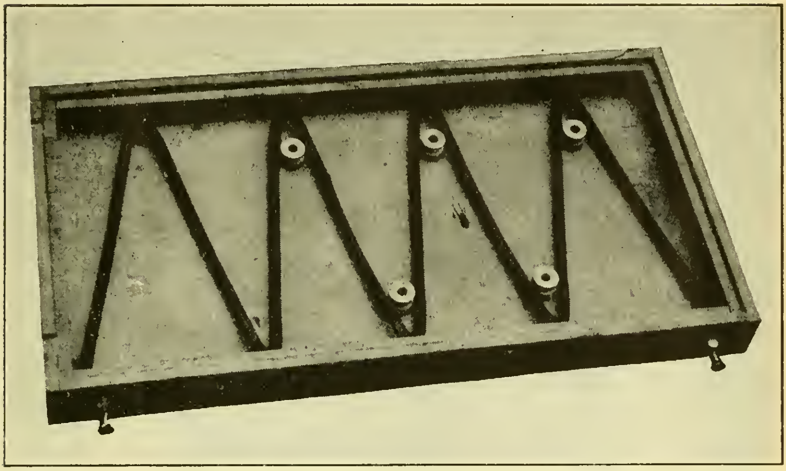


FIG. 28.—ASBESTOS BOARD TRAY AND COIL OF NICHROME WIRE TO SUPPORT AND HEAT WATER-BATHS.

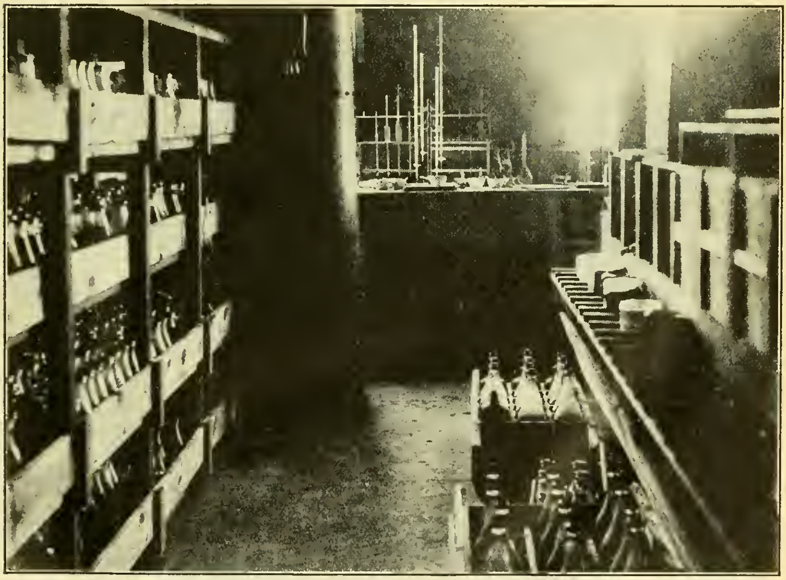


FIG. 29.—VIEW IN CHEMICAL LABORATORY, SHOWING SAMPLE STORAGE, BOTTLE-DRAINING RACKS, BOTTLE CARRIERS, AND OTHER EQUIPMENT.

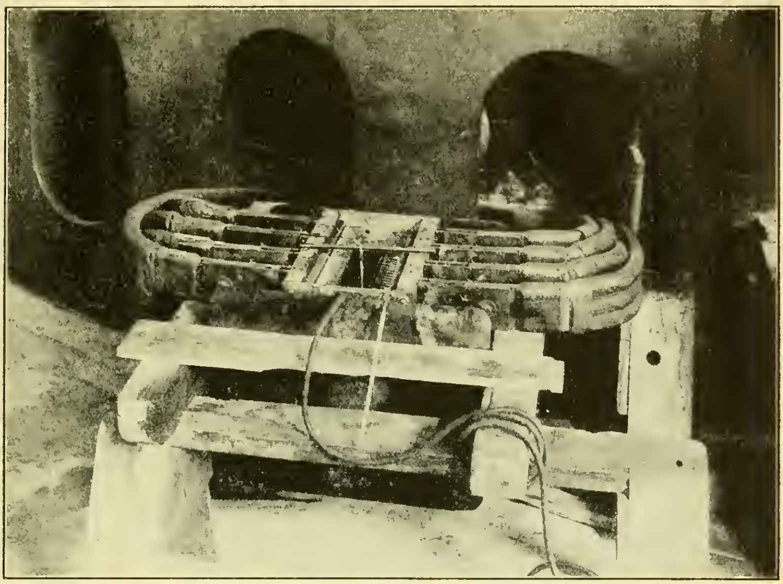


FIG. 30.—10 000-AMPERE SHUNT, DESIGNED BY MR. I. B. SMITH, OF THE LEEDS AND NORTHRUP COMPANY.

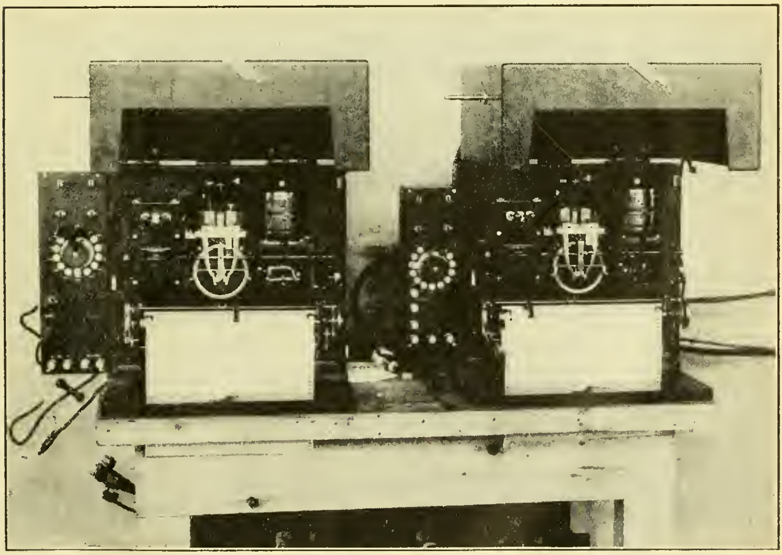


FIG. 31.—VOLTMETER AND AMMETER POTENTIOMETERS FOR MEASURING DIRECT-CURRENT POWER ON TESTS.

PART IV.

THE TESTS.

98.—Tests on Unit No. 13.—The tests on Unit No. 13 may be divided into three series. The first is indicated by letters instead of numbers. The second comprises the tests bearing numbers from 1 to 25, inclusive, and the third comprises all the remaining numbers. Certain tests which were not turbine tests have been given letters, or numbers, and these letters or numbers, therefore, are not in the tables or on the diagrams connected with the turbine tests proper. In all cases the tabulations of turbine tests give them in consecutive order, whether the numbers, or letters, appear consecutively or otherwise.

The first series (letters) comprises the preliminary tests which were made while the corps of observers was becoming familiar with its work, and before the high-precision electric instruments were inserted in the circuit. In this series the tail-race samples were taken from the lower, or stop-log-slot, sampling section, excepting Samples 41 and 42, which were from the centers of the upper and lower draft-tubes on the upper sampling section, which was a vertical section in the plane of the Pitot tube pipes extending into the tail-race just down stream from the discharge ends of the draft-tubes. Sample 43 was usually taken from the center of the upper sampling pipe on the lower section, but could be taken from any other position in the tail-race.

The tests of the second series (Nos. 1 to 25) were executed in a manner similar to those of the first, but were made immediately after inserting the high-precision electric measuring equipment, and, therefore, are to be regarded as more accurate than those of the first series.

The tests of the third series were made after moving the tail-race sampling system from the lower to the upper sampling section, and after increasing the number of regular tail-race samples from 12 to 18, but cutting out the extra samples, Nos. 41, 42, and 43.

The lettered tests, therefore, relate to preliminary work, before accurate power measurements were made, and the numbered tests relate to those in which accurate power records were taken by high-precision instruments. The tests bearing numbers higher than 25 were those during which the tail-race samples were taken from the upper sampling section, and are the most accurate of any.

The object in moving the tail-race sampling system from the lower to the upper section, after the second series, was to secure more reliable samples and a larger number of them. The principal reason for this change was the fact that the risers leading to the sampling pumps had to pass up through several feet of water which was more or less diluted by water from other turbine units not being tested. Should any of these risers leak it would be likely to take in water which did not contain a sufficient quantity of salt or, at least, did not come from the location at which it was intended to secure a sample.

99.—Tests on Unit No. 12.—There was only one series of tests on Unit No. 12, numbered from 200 to 212, inclusive. They were executed in consecutive order, from September 28th to October 13th, 1914, and cannot be considered as good as the third series on Unit No. 13. This is due to the less favorable weather conditions, to changes in personnel of observers and manipulators, and also to the fact that the hydraulic conditions of the tests were not so uniform and satisfactory. In particular, the power measurements are not as satisfactory as those on Unit No. 13, and this is probably due to the fact that the power was observed and the electrical instruments manipulated in the latter case by Mr. I. B. Smith, the designer of the instruments.

100.—Tests on Unit No. 11.—There were two series of tests on Unit No. 11. The tests were started with No. 105, on September 12th, but after running four more tests, ending with No. 109, on September 18th, it became necessary to shut down the unit for repairs, and there were no more until November 9th, when Test 111 was made. This second series was then continued consecutively up to 122 on November 21st, which ended the series, and in fact all the turbine tests.

The generator of this unit was of the alternating-current type, and those of Units Nos. 12 and 13 were both of the direct-current type. Consequently, an entirely new set of electrical measuring instruments was required, with additional observers, and correspondingly more difficulty in securing data. The power records, however, are very consistent, under the circumstances, and approach in accuracy those of the best tests on Unit No. 13.

101.—Field Data.—Tables 46, 47, 48, and 49,* contain a summary of all the field data observed during the entire series of turbine tests.

* These tables are not reproduced in this paper, but are filed in the Library of the Society for reference by any one interested.

exclusive of the data necessary for the chemical work proper. As a general rule, all instrumental readings given on these sheets are averages computed from numerous readings taken at the end of equal intervals of time, with the maximum variation during the entire test recorded immediately after the corresponding averages.

The method of measuring the head was specified in the contract, but a slight divergence therefrom was agreed on, owing to the difficulty of measuring the head-water exactly 5 ft. down stream from the racks at the water surface. This difficulty arose from the fact that the gauge-boards would have been somewhat inconveniently situated at that exact distance in all the tests, as the distance to the gauge-board from the racks at the water level varied considerably with changes in elevation of head-water due to the slope of the racks. At surface elevation 197 the location of the gauge-boards was just about as specified in the contract.

The principal head-water readings, however, were made by measuring downward at three points across the head-race from an angle iron which was fixed at Elevation 204. The scales on the measuring rods were inverted, with the 204-mark at the bottom of the rod, so that, by holding the bottom of the rod in contact with the water surface and reading the inverted scale on the rod at the edge of the angle iron, the true elevation of the water surface was ascertained without further correction.

As a matter of convenience, the rods passed through holes in the flat side of the angle, so that by boring holes the size of a nail at various points along the rod and then inserting a nail in the hole nearest to the proper graduation mark, the rod could be left by the gauge reader to swing loosely at about the correct elevation, thus being ready for use at any time without the necessity for much adjustment.

The three rods and the two gauge-boards gave the elevations of the water surface at five points in the head-race. The distances between these five points were not exactly equal across the race, owing to the desire to obtain readings of the two rods adjacent to the center rod, which would indicate the surface elevation back of the two rack girders where the water surface was relatively low.

The tail-water elevation was observed in much the same manner, there being two gauge-boards at the side-walls and three rods manipulated in a manner similar to that used in the head-water readings. The

three rods, however, were just outside the power-house, in the center-line of the three openings of the tail-race formed by the two side-walls and the two tail-gate girders. The rods, therefore, were equally spaced on each side of the rod, at the center.

There was also a float-gauge in the head-race, at the inside of the west rack girder, and one in the tail-race at the west wall. These floats did not give accurate results because of the influence of the velocity heads created in and around the gauge-board box. Probably more accurate results could have been obtained by a further study and re-arrangement of the float-gauges.

The output of Units Nos. 13 and 12 was measured by three different sets of direct-current equipment. The first set consisted of the ordinary switch-board instruments; the second consisted of high-precision instruments purchased from the Leeds and Northrup Company, and the third was a Weston outfit, furnished and operated by the Allis-Chalmers Company for its own information.

In the tests of Unit No. 11 there were also three sets of instruments. The current in this case, however, was 25-cycle, at about 2 200 volts, thus limiting the tests to a fixed speed which exceeded 100 rev. per min.

The first set was the switch-board equipment; the second was a high-precision Weston outfit purchased for the purpose, and the third was the Allis-Chalmers' Weston outfit.

The speed of the turbine was taken by a tachometer, and checked by actual count.

The tables show the data for the salt solution, the temperature of the solution in the tank, the concentration computed from the hydrometer reading, the two pairs of divisions on the gauge scale between which the surface of the solution stood at the start or finish of each test, the time of passing these divisions, the time of starting the salt-solution discharge, the time when the discharge was adjusted and uniform, and the time of starting and finishing the test.

The pressures at which the distributing pipes were operated are given, but it must be understood that these are mere readings of gauges on the sides of the pipes. These gauges had not been calibrated, and were subject to the usual errors when attempting to secure pressures from pipes inserted in the side of another pipe in which the pressures are to be estimated. The gauges, however, served their pur-

pose well in supplying a means for controlling the distribution of salt in the head- and tail-races.

There is also a column for remarks at the right of each of the data tables.

102.—Tables of Observed Concentrations.—Space will not be devoted to the detail of all the concentrations observed in all the tests. If it is desired to examine the detail of any test, which is not fully supplied in this paper, the original data are on file, and access will be furnished by the writer.

The four principal series of titrations, however, given in detail in the next four sections, furnish a complete exhibit of the best of the chemical work.

In the case of Units Nos. 12 and 13, and the first series of tests on Unit No. 11, the strength of the silver nitrate solution was about 1.56 grammes per liter. In the case of the second series of tests on Unit No. 11, in fact, the last of the turbine tests, the strength was about 1.45 grammes per liter, resulting approximately from the solution of 1.45 grammes in 1 liter of distilled water.

As a general rule, the tests were titrated the day after they were made, the evaporations having been made over night in the laboratory, which operated continuously. The second treatments of certain of the tests, usually indicated by the letter *B* affixed to the regular test number, are exceptions to this rule. They were made long after the first treatments, and the results show that the samples suffered more or less from reductions of volume due to evaporation during the interval.

This experience indicates that extraordinary care must be taken to preserve samples, when they are to be kept for any length of time before treatment. One-quart large-mouthed cream bottles with paraffined cork stoppers will not do for this purpose.

103.—Concentrations Observed During Eleven Consecutive Tests on Unit No. 13.—The dilute salt solution is prepared by diluting one 10-c.c. pipette of salt solution to the graduation mark of a 1-liter flask with distilled water, 10 c.c. of this dilution then being taken for titration. In all cases $\frac{1}{2}$ liter of normal head-water and $\frac{1}{2}$ liter of tail-water are evaporated to about 10 c.c. in a casserole for titration. Titrations in parentheses have been omitted from the calculations as doubtful. The unit of titration is $\frac{1}{100}$ c.c., all decimal points being omitted except in the final averages, where the unit is 1 c.c. The

capacities of the pipettes and flasks mentioned are nominal; the actual capacities may be determined by Figs. 4 and 6. In all cases, Pipette No. 18 791 was used on strong salt solution, and Pipette No. 21 047 on dilute salt solution; and Flasks 1 456 and 1 476 were used on dilution of salt solution. Flask No. 11 was used for measuring normal head- and tail-water in all cases except Test 32, on which Flask No. 3 was used. Missing test numbers represent tests which were not chemical. The chemical tests follow in the consecutive order in which they were made.

Concentrations Observed During Eleven Tests on Unit No. 13.

Titrations in parentheses have been omitted as doubtful. The unit of titration is $\frac{1}{100}$ c.c., all decimal points being omitted except in final averages, where the unit is 1 c.c.

Test 26, August 26th, 1914.

Titration. Nitrate (1.56 Grammes). August 24th

Dilute salt solution. Temp. 19½° cent.		Normal head-water. Temp. 18° cent.	
5 280	5 280	1 310	1 305
5 290	5 290	1 305	1 306
5 290	5 288	(1 330)	1 295
5 287	5 285	1 300	1 295
5 285	5 283	1 282	1 310
5 282	5 290	1 295	(1 325)
Mean = 52.858		Mean = 13.003	

Tail-water Titrations. Temp. 18½° cent.

5 810		5 813		5 770	
	5 871		5 805		5 860
5 840		5 925		5 920	
	5 365		5 440		5 481
5 425		5 380		5 465	
	5 395		5 408		5 420

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper....	58 518	58 388	58 46
Lower....	53 912	54 428	54.199
Averages.	56.215	56.408	56.329 = Arith. mean.
Weighted mean = 56.276			

Test 32, August 26th, 1914.

Titration. Nitrate (1.56 Grammes). August 24th.

Dilute salt solution. Temp. 19.8° cent.		Normal head-water. Temp. 19° cent.	
5 248	5 257	1 302	1 305
5 260	5 245	1 310	1 313
5 235	5 260	1 290	1 292
5 255	5 254	1 310	1 302
5 260	5 253	1 300	1 300
5 255	5 262	1 295	1 300
Mean = 52.537		Mean = 13.016	

Tail-Water Titrations. Temp. 19° cent.

5 290		5 360		5 405	
	5 405		5 410		5 470
5 290		5 400		5 450	
	5 325		5 367		5 395
5 320		5 380		5 350	
	5 460		5 370		5 392

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	53 490	54 337	53.867
Lower.....	53 712	53 748	53.732
Averages..	53.601	54.042	53.799 = Arith. mean.
Weighted mean = 53.776			

Test 33, August 27th, 1914.

Titration. Nitrate (1.56 Grammes). August 24th.

Dilute salt solution. Temp. 19.8° cent.		Normal head-water. Temp. 21° cent.	
5 342	5 345	1 290	1 280
5 348	5 350	1 308	1 300
(5 315)	5 350	1 288	1 293
5 340	5 340	1 293	1 282
5 337	5 340	1 305	1 290
5 340	5 340	1 290	1 293
Mean = 53.429		Mean = 12.927	

Tail-water Titrations. Temp. 21.8° cent.

5 570		5 637		5 740	
	5 685		5 657		5 870
5 700		5 730		5 715	
	5 510		5 568		5 608
5 585		5 558		5 597	
	5 560		5 550		5 592

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper....	56 644	57 455	57.004
Lower....	55 526	55 830	55.698
Averages.	56.085	56.642	56.351 = Arith. mean.
Weighted mean = 56.299			

Test 39, August 28th, 1914.

Titration. Nitrate (1.56 Grammes). August 24th.

Dilute salt solution. Temp. 23.8° cent.		Normal head-water. Temp. 19° cent.	
5 390	5 390	1 297	1 310
5 380	5 388	1 315	1 310
5 400	5 400	1 325	1 325
5 400	5 395	1 300	1 300
5 400	5 398	1 298	1 300
5 400	5 400	1 295	1 310
Mean = 53.951		Mean = 13.071	

Tail-water Titrations. Temp. 19° cent.

5 847		5 990		5 880	
	6 160		5 995		6 060
6 190		6 160		6 135	
	5 333		5 335		5 465
5 330		5 305		(5 070)	
	5 365		5 325		5 340

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper....	60 694	60 175	60.463
Lower....	53 332	53 663	53.498
Averages.	57.013	56.622	57.185 = Arith. mean.
Weighted mean = 56.968			

104.—Concentrations Observed During the Tests on Unit No. 12.— The dilute salt solution is prepared in the same way as described in Section 103. In these tests ½ liter of normal head-water, ½ liter of tail-water and ½ liter of special dilution were evaporated to about 10 c.c. in a casserole for titration. Titrations in parentheses have been omitted as doubtful. The unit of titration is $\frac{1}{100}$ c.c., all decimal points being omitted except in the final averages, where the unit is 1 c.c. The capacities of the pipettes and flasks mentioned are nominal; the actual capacities may be determined by Figs. 4 and 6. In all cases, Pipette No. 18 791 was used on strong salt solution, and Pipette No. 21 047 on

dilute salt solution, or stock solution for special dilutions. Flasks were used as indicated in the record of each test.

Test 40, August 28th, 1914.

Titration.		Nitrate (1.56 Grammes). August 24th.	
Dilute salt solution. Temp. 21° cent.		Normal head-water. Temp. 22° cent.	
5 330	5 330	1 340	1 333
5 327	5 339	(1 425)	1 325
5 343	5 325	1 328	1 298
5 320	5 335	1 320	1 310
5 330	5 333	1 325	1 310
5 330	5 332	1 312	1 290
Mean = 53.312		Mean = 13.174	

Tail-water Titrations. Temp. 20.5° cent.

(5 320) 5 455 5 610 5 573 5 650	5 555 5 550 5 570 5 610
5 460 5 480 5 495 5 500	5 500 5 600 5 553 5 515 5 570

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	55 720	55 712	55.716
Lower......	54 838	55 476	55.192
Averages..	55.279	55.594	55.439 = Arith. mean.
Weighted mean = 55.402			

Test 41, August 31st, 1914.

Titration. September 3d.		Nitrate (1.56 Grammes). August 31st.	
Dilute salt solution. Temp. 24.5° cent.		Normal head-water. Temp. 20° cent.	
5 135	5 150	1 300	1 315
5 140	5 147	1 302	1 325
5 160	5 155	1 317	1 330
5 135	5 140	1 320	1 315
5 163	5 140	1 330	1 330
5 150	5 150	1 320	1 319
Mean = 51.471		Mean = 13.186	

Tail-water Titrations. Temp. 20° cent.

5 320 5 392 5 445 5 345 5 470	5 440 5 385 5 494 5 485
5 385 5 405 5 380 5 390	5 405 5 325 5 430 5 420 5 455

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	53 944	5 451	54.196
Lower.....	5 390	5 407	53.994
Averages..	53.922	54.290	54.095 = Arith. mean.
Weighted mean = 54.063			

105.—Concentrations Observed During the First Series of Tests on Unit No. 11.—The dilute salt solution is prepared by diluting one 10-c.c. pipette of salt solution to the graduation mark of a 1-liter flask with distilled water, 10 c.c. of this dilution then being taken for titration, except in Tests 108 and 109, where four pipettes of salt solution were diluted to 3 liters. In all cases ½ liter of normal head-water, ½ liter of tail-water and ½ liter of special dilution are evaporated to about 10 c.c. in a casserole for titration. Titrations in parentheses have been

omitted as doubtful. The unit titration is $\frac{1}{100}$ c.c., all decimal points being omitted except in final averages, where the unit is 1 c.c. Capacities of pipettes and flasks mentioned are nominal; the actual capacities may be determined by Figs. 4 and 6. In all cases Pipette No. 18 791 was used on strong salt solution, and Pipette No. 21 047 on dilute salt solution, or stock solution for special dilutions. Flasks were used as indicated in the record of each test.

TEST 42, AUGUST 31ST, 1914.

TITRATION. Dilute salt solution. Temp. 29° cent.		NITRATE (1.56 GRAMMES). AUGUST 31ST. Normal head-water. Temp. 21½° cent.	
5 465	5 465	1 320	1 320
5 455	5 455	1 300	1 320
5 465	5 470	1 345	1 335
5 455	5 460	1 315	1 320
5 460	5 463	1 315	1 305
5 465	5 455	1 300	1 300
Mean = 54.611		Mean = 13.163	

TAIL-WATER TITRATIONS. Temp. 20.5° cent.

5 485		5 590	-	5 630	
	5 620		5 620		5 685
5 515		5 660		5 693	
	5 650		5 585		5 705
5 587		5 590		5 625	
	5 590		5 665		5 650

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper....	55 740	56 570	56.109
Lower....	56 042	56 460	56.274
Averages.	55.891	56.515	56.192 = Arith. mean.
Weighted mean = 56.127			

TEST 43, SEPTEMBER 1ST, 1914.

TITRATION. SEPTEMBER 3D. 4TH. Dilute salt solution. Temp. 26° cent.		NITRATE (1.56 GRAMMES). AUGUST 31ST. Normal head-water. Temp. 25° cent.	
5 425	5 420	1 310	(1 260)
5 430	5 425	1 307	1 300
5 422	5 423	1 315	1 305
5 425	5 425	1 320	1 315
5 425	5 425	1 312	1 315
5 425	5 425	1 305	1 315
Mean = 54.246		Mean = 13.108	

TAIL-WATER TITRATIONS. Temp. 23.5° cent.

5 530		5 620		5 780	
	5 730		5 690		5 880
5 670		5 815		5 785	
	5 400		5 490		5 535
None*		5 420		5 510	
	5 390		5 480		5 490

* Bottle broken.

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	56 730	57 838	57.222
Lower.....	54 033	55 010	54.644
Averages..	55.382	56.424	56.009 = Arith. mean.
Weighted mean = 55.786			

106.—Concentrations Observed During the Second Series of Tests on Unit No. 11.—The dilute salt solution is prepared by diluting one

10-c.c. pipette of salt solution to the graduation mark of a 1-liter flask with distilled water, 10 c.c. of this dilution then being taken for titration. In all cases ½ liter of tail-water and ½ liter of special dilution are evaporated to about 10 c.c. in a casserole for titration. Titrations in parentheses have been omitted as doubtful. The unit titration is $\frac{1}{100}$ c.c., all decimal points being omitted except in the final averages, where the unit is 1 c.c. Capacities of pipettes and flasks mentioned are nominal; the actual capacities may be determined by Figs. 4 and 6. Flasks Nos. 1 407 and 1 457 were used for dilution of salt solution on all tests except Test 117, in which Flasks Nos. 1 407 and 1 476 were used.

Test 44, September 1st, 1914.

Titration.		Nitrate (1.56 Grammes). August 31st.	
Dilute salt solution. Temp. 28° cent.		Normal head-water. Temp. 23° cent.	
5 445	5 440	1 300	1 295
5 440	5 435	1 310	1 282
5 450	5 445	1 300	1 310
5 450	5 445	1 287	1 305
5 450	5 447	1 300	1 300
5 450	5 445	1 280	1 300
Mean = 54.452		Mean = 12.974	

Tail-Water Titrations. Temp. 23° cent.

5 475		5 675		5 745	
	5 755		5 685		5 875
5 633		5 845		5 900	
	5 655		5 645		5 765
5 650		5 644		5 742	
	5 615		5 670		5 730

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper....	56 766	58 012	57.320
Lower....	5 641	57 104	56.795
Averages.	56.588	57.558	57.058 = Arith. mean.
Weighted mean = 56.952			

Test 45, September 2d, 1914.

Titration.		Nitrate (1.56 Grammes). August 31st.	
Dilute salt solution. Temp. 22° cent.		Normal head-water. Temp. 25.3° cent.	
5 450	5 460	1 303	1 300
5 460	5 460	1 300	1 300
5 450	5 472	1 296	1 294
5 450	5 465	1 286	1 294
5 456	5 450	1 290	1 290
5 452	5 455	1 290	1 290
Mean = 54.567		Mean = 12.944	

Tail-Water Titrations. Temp. 24° cent.

5 362		5 450		5 555	
	(5 085)		5 446		5 590
5 427		5 545		5 565	
	5 394		5 472		5 640
5 365		5 400		5 485	
	5 380		5 440		5 475

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper....	5 446	55 390	54.925
Lower....	53 848	55 024	54.501
Averages..	54.154	55.207	54.701 = Arith. mean.
Weighted mean = 54.549			

TEST 46, SEPTEMBER 2D, 1914.

TITRATION, SEPTEMBER 4th. Dilute salt solution Temp. 19.5° cent.		NITRATE (1.56 GRAMMES). AUGUST 31st. Normal head-water. Temp. 21.5° cent.	
5 122	5 133	1 295	1 297
5 135	5 133	1 291	1 275
5 130	5 132	1 275	1 275
5 130	5 130	1 270	1 270
5 140	5 132	1 270	1 303
5 140	5 135	1 274	1 285
Mean = 51.327		Mean = 12.817	

TAIL-WATER TITRATIONS. Temp. 21.5° cent.

5 540		5 649		5 590	
	5 672		5 592		5 690
5 575		5 775		5 770	
	5 370		5 394		5 415
5 335		5 422		5 395	
	5 435		5 428		5 410

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	56 422	56 605	56.503
Lower.....	53 905	54 084	54.004
Averages..	55.164	55.344	55.254 = Arith. mean.

Weighted mean = 55.227

Flasks Nos. 1 407 and 1 457 were used for special dilutions in all tests. Pipettes Nos. 9 171 and 9 206 were used on strong salt solution, dilute salt solution, and stock solution. No normal head-water was evaporated. The methods of making the solutions are explained in the rules for the chemical laboratory at the end of Part II. See Sections 88 and 89.

107.—Discharge Computations.—Tables 50, 51, 52, and 53 (Plates LVI, LVII, LVIII, and LIX), contain all the data necessary for the chemical determination of the discharge in the various tests. These calculations have been carried out with more refinement, in certain respects, and more elaborately, than necessary. This is especially true when these methods are compared with the simple one of weighing samples, explained in Sections 10 to 18, and applied in Section 78 to the determination of the capacity of the iron calibrator.

In view of the full explanation of the theories and equations given in Parts I and II, it will not be necessary to enter into a detailed explanation of the nature of the discharge computations, which are, in fact, quite fully explained by the tables.

It may be repeated that the tests are entered in consecutive order in the tables exactly as they were made in the field, the missing numbers,

Test 200, September 28th, 1914.

Titration, September 29th.
Nitrate (1.56 grammes), September 25th.

Dilute salt solution. Temp. 15.75° cent. Flask No. 1 456.		Normal head-water. Temp. 15° cent. Flask No. 11.	Special dilutions.* Temp. 15° cent. Flask No. 1 456.		
4 685	4 675	1 300	4 350	4 316	8 666
4 687	4 685	1 288	4 410	4 324	8 734
4 687	4 685	1 308	4 377	4 345	8 722
4 677	4 670	1 290	4 335	4 350	8 685
4 675	4 680	1 290			
4 683	4 671	1 310			
Mean = 46.800		Mean = 12.977	Mean = 43.509		

Tail-Water Titrations.
Temp. 15.5° cent. Flasks Nos. 3-11.

4 420		4 390		4 587	
	4 454		4 525		4 614
4 516		4 510		4 575	
	4 830		5 337		5 345
5 130		†5 465		5 540	
	5 447		5 610		5 540

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	44 580	45 752	45.101
Lower.....	52 180	54 744	53.604
Averages.	48.380	50.248	49.353 = Arith. mean.

Weighted mean = 48.962.

* 10 c.c. salt solution from each of Samples ———, 40 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† Average of the two values to the right and the lower one on the left. Sample pipe was broken.

Test 201, September 29th, 1914.

Titration. October 1st.
Nitrate (1.56 grammes), September 9th.

Dilute salt solution. Temp. 15.5° cent. Flasks Nos. 1 456-1 476.		Normal head-water. Temp. 14.5° cent. Flask No. 11.	Special dilutions.* Temp. 15° cent. Flasks Nos. 1 456-		
5 150	5 148	1 295	4 958	4 932	9 890
5 149	5 148	1 295	4 937	4 901	9 838
5 145	5 145	1 300	4 875	4 920	9 795
5 145	5 145	(1 250)	4 870	4 915	9 785
		1 290			
Mean = 51.469		Mean = 12.950	Mean = 49.135		

Tail-Water Titrations.
Temp. 15° cent. Flask No. 3.

5 297		5 310		5 382	
	5 301		5 386		5 375
5 297		5 310		5 382	
	5 455		5 615		5 580
5 400		† 5 562		5 607	
	5 455		5 615		5 580

Calculations of Averages for Quarters and Halves of Race.

	West	East.	Averages.
Upper	53 030	53 813	53.378
Lower	54 680	55 994	55.410
Averages..	53.855	54.904	54.394 = Arith. mean.

Weighted mean = 54.140.

Note: Depths 1 and 3 averaged chemically.
" 4 and 6 " "

* 10 c.c. salt solution from each of Samples ———, 30 c.c., diluted with 2 100 c.c. normal head-water, and 10 c. c. of this diluted to 1 000 c. c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† Average of the two values to the right and the lower one on the left. Sample pipe was broken.

TABLE 50.—DATA FOR COMPUTATION OF TURBINE DISCHARGE. COMPILED FROM RECORDS OF TESTS MADE ON UNIT NO. 13, JULY 18TH, 1914, TO AUGUST 19TH, 1914.—SEE NOTE BELOW.

Line.	UNIT No. 13.	Formula	Units	Symbol.
1	Temperature: Salt solution in tank during test		°C	
2	Temperature: Salt solution when pipetted in laboratory for dilution with distilled water		°C	
3	Temperature: Distilled water for diluting salt solution in laboratory		°C	
4	Temperature: Normal head-water when measured for evaporation in laboratory		°C	
5	Temperature: Tail water when measured for evaporation in laboratory		°C	
6	Temperature: Head-water in canal		°C	
7	Temperature: Tail-water in tail-race		°C	
8	Mean temperature of head and tail water during tests	(Line 6 + Line 7) ÷ 2	°C	
9	Number of pipette used in measuring out salt solution for dilute salt solution			
10	Volume of salt solution discharged by pipette for dilute salt solution at temperature of Line 2		c.c	
11	Numbers of flasks used in measuring distilled water			
12	Volume of flask measuring distilled water at temperature of Line 3		c.c.	
13	Number of pipette used in measuring out dilute salt solution			
14	Volume of dilute salt solution discharged by pipette at temperature of Line 3		c.c.	
15	Titration per pipette of dilute salt solution sample in laboratory		c.c.	
16	Titration per liter of dilute salt solution in laboratory at temperature of Line 3	Line 15 ÷ Line 14	c.c.	
17	Ratio of dilution of salt solution with distilled water	Line 13 ÷ Line 10		
18	Titration per liter of salt solution at temperature of Line 2	Line 16 × Line 17	c.c	
19	Concentration of silver nitrate, grammes per liter, approximately		gms	
20	Concentration (approximate) in grammes per liter of salt solution at temperature of Line 2		gms	
21	Approximate strength of solution, in percentage, at temperature of Line 2	$p = \frac{c}{d}$ (from curves)		
22	Density: Salt solution in tank during test at temperature of Line 1			
23	Density: Salt solution when pipetted for dilution with distilled water in laboratory at temperature of Line 2			
24	Density: Salt solution at mean temperature of head and tail water, temperature of Line 8			
25	Titration of salt solution at mean temperature of head and tail-water, temperature of Line 8 (per liter)	Line 18 × Line 24 ÷ Line 23	c.c.	v
26	Gauge reading on salt solution tank at start			
27	Gauge reading on salt solution tank at end			
28	Total time, in seconds, elapsed between divisions at start and end		sec.	
29	Rate of discharge; actual	$\frac{8.466 \times (\text{Line } 27 - \text{Line } 26)}{\text{Line } 28}$	c.f.s.	
30	Equivalent rate of discharging salt solution at mean temperature of head and tail-water, temperature of Line 8	Line 29 × Line 22 ÷ Line 24	c.f.s.	q
31	Numbers of flasks used in measuring normal head-water samples			
32	Average volume of flasks measuring normal head-water samples at temperature of Line 4		c.c	
33	Numbers of flasks used in measuring dosed tail water samples			
34	Average volume of flasks measuring dosed tail water samples at temperature of Line 5		c.c.	
35	Density: Normal head-water when measured in laboratory at temperature of Line 4			
36	Density: Normal head water at mean temperature of head and tail-water, temperature of Line 8			
37	Density: Dosed tail water when measured in laboratory at temperature of Line 5			
38	Density: Dosed tail-water at mean temperature of head and tail-water, temperature of Line 8			
39	Volume of normal head-water samples at mean temperature of head and tail water, temperature of Line 8	Line 32 × Line 36 ÷ Line 35	c.c.	
40	Titration of normal head-water sample in laboratory		c.c.	
41	Titration per liter of normal head-water sample at mean temperature of head and tail-water, temperature of Line 8	Line 40 ÷ Line 39	c.c.	
42	Correction to titration of normal head-water per liter		c.c	
43	Correct titration of normal head-water at mean temperature of head and tail-water, temperature of Line 8	Line 41 − Line 42	c.c.	v_1
44	Volume of tail-water sample at mean temperature of head and tail water	Line 34 × Line 37 ÷ Line 38	c.c	
45	Titration of dosed tail-water sample at temperature of dosed tail water when measured for evaporation, temperature of Line 5		c c	
46	Titration per liter of dosed tail-water at mean temperature of head and tail-water	$\frac{\text{Line } 45 \times \text{Line } 38}{\text{Line } 44 \times \text{Line } 37}$	c.c.	v_2
47	Approximate value of k			k
48	Value of $k v_2$	Line 46 × Line 47		kv_2
49	Value of $v - k v_2$	Line 25 − Line 48		$v - kv_2$
50	Value of $v_2 - v_1$	Line 46 − Line 43		$v_2 - v_1$
51	Value of r	Line 49 ÷ Line 50		r
52	Value of $Q_1 = qr$	Line 30 × Line 51	c.f.s.	qr
53	Value of kq	Line 30 × Line 47		kq
54	Value of $Q_2 = Q_1 + kq$	Line 52 + Line 53	c.f.s.	Q_2

TESTS.

Line	G	H	I	K	L	N	O	S
1	18.2	18.1	18.3	18.3	18.2	17.8	17.8	18.8
2	21.0	20.0	21.0	25.0	25.0	22.5	22.0	25.0
3	21.0	21.0	21.0	24.0	25.0	22.5	22.0	26.0
4	21.0	20.0	21.0	22.0	23.5	20.0	22.0	23.5
5	21.0	20.0	21.0	21.0	23.5	20.0	22.5	23.5
6	19.1	19.3	19.4	19.4	19.4	19.0	18.6	18.3
7	19.1	19.3	19.4	19.4	19.4	19.0	18.6	18.3
8	19.1	19.3	19.4	19.4	19.4	19.0	18.6	18.3
9	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791
10	9.960	9.979	9.980	9.963	9.951	9.982	9.962	9.965
11	1 456	1 470	1 476	1 476	1 476	1 474, 1 479	1 476	1 456
12	1 002.87	1 003.50	1 003.50	1 003.60	1 003.63	1 003.00	1 003.55	1 003.00
13	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047
14	10.011	10.012	10.013	10.017	10.018	10.015	10.016	10.020
15	54.650	54.504	54.470	55.165	55.229	53.512	52.608	54.373
16	5 458.50	5 443.90	5 440.80	5 509.10	5 513.10	5 343.20	5 252.90	5 418.40
17	101.40	100.56	100.55	100.65	100.82	100.54	100.54	100.45
18	548 580	547 410	547 070	553 630	554 170	537 210	528 190	547 000
19	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622
20	294.7	294.8	294.2	297.3	297.8	288.7	281.0	294.2
21	24.85	24.63	24.63	25.09	25.12	24.43	24.08	24.87
22	1.1674	1.1672	1.1672	1.1693	1.1696	1.1644	1.1614	1.1674
23	1.1661	1.1664	1.1658	1.1682	1.1682	1.1631	1.1601	1.1657
24	1.1670	1.1666	1.1666	1.1688	1.1691	1.1638	1.1611	1.1675
25	548 060	547 560	547 450	553 650	553 800	538 230	528 870	546 640
26	8	6	30	6	43	10	13	12
27	50	31	63	5[illegible]	64	72	71	62
28	1 519.9	972.4	1 069.0	1 127.0	981.1	2 428.0	2 300.1	1 618.0
29	0.23394	0.20034	0.20137	0.22530	0.20661	0.21619	0.20902	0.23394
30	0.23401	0.20034	0.20147	0.22545	0.20680	0.21634	0.20908	0.23282
31	4 & 6	6 & 4	1 476	1 456	1 474	1 474	2 & 10	1 476-3
32	1 474, 1 478; 1 003.55	1 476, 1 474; 1 003.55	1 003.55	1 002.90	1 003.70	1 003.61	1474-1476; 1 003.6	1 003.50; 500.44
33	9 & 5; 2 & 11	11	11	11	3	11	3	11
34	500.61	500.60	500.61	500.61	500.45	500.60	500.43	500.64
35	0.9980	0.9983	0.9980	0.9977	0.9973	0.9983	0.9975	0.9973
36	0.9985	0.9984	0.9984	0.9984	0.9984	0.9985	0.9984	0.9984
37	0.9981	0.9984	0.9981	0.9981	0.9973	0.9983	0.9976	0.9973
38	0.9985	0.9985	0.9985	0.9983	0.9985	0.9985	0.9987	0.9987
39	1 003.1	1 003.4	1 003.1	1 002.2	1 002.6	1 003.1	1 002.5	1 002.8; 499.79; 1 001.0
40	25.365	25.389	25.370	25.410	25.388	25.083	25.383	25.632
41	25.287	25.303	25.301	25.364	25.322	24.998	25.330	25.609
42	−0.263	−0.263	−0.263	−0.263	−0.263	−0.263	−0.263	−0.263
43	25.024	25.040	25.038	25.121	25.059	24.735	25.067	25.346
44	500.31	500.30	500.41	500.41	499.85	500.50	499.84	499.94
45	51.176	43.038	43.047	49.761	47.172	45.153	43.878	51.540
46	105.329	81.087	91.255	99.479	94.186	90.233	87.880	103.148
47	0.9755	0.9758	0.9757	0.9753	0.9753	0.9762	0.9767	0.9753
48	99.822	88.807	89.038	97.022	92.150	88.085	85.822	100.598
49	548 560	547 461	547 361	534 953	535 498	538 143	528 781	545 769
50	77.905	55.997	66.217	74.358	69.427	65.498	62.812	77.902
51	7 009.98	8 295.34	8 266.17	7 463.36	7 909.88	8 216.10	8 419.52	7 043.97
52	1 661.6	1 661.9	1 665.4	1 662.6	1 647.1	1 776.7	1 767.7	1 640.0
53	0.228	0.165	0.197	0.220	0.201	0.211	0.205	0.227
54	1 661.7	1 662.1	1 665.6	1 662.6	1 647.3	1 776.9	1 767.9	1 640.9

Line	T	U	V	W	6	7	8
1	18.4	16.5	15.0	17.6	18.0	17.6	17.5
2	21.0	20.0	23.5	21.6	25.0	23.0	22.0
3	25.5	20.0	23.5	21.6	25.0	23.0	22.0
4	25.0	25.0	19.0	19.0	22.6	25.0	20.0
5	24.0	25.0	19.0	19.0	21.6	20.0	20.0
6	18.5	18.8	18.5	18.6	19.0	19.0	19.0
7	18.6	18.8	18.3	19.0	19.6	19.0	19.0
8	18.6	18.6	18.4	18.8	19.22	19.0	19.0
9	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791
10	9.983	9.980	9.963	9.981	9.965	9.961	9.981
11	1 456, 1 476	1 456, 1 476	1 460, 1 470	1 470, 1 476	1 456, 1 476	1 456, 1 476	1 456, 1 476
12	1 003.83	1 003.50	1 003.28	1 003.21	1 003.90	1 003.22	1 003.29
13	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047
14	10.020	10.012	10.017	10.014	10.018	10.015	10.016
15	56.108	53.485	54.490	53.378	62.836	54.622	54.533
16	5 599.00	5 337.10	5 439.80	5 330.40	6 274.10	5 454.00	5 445.10
17	100.50	100.55	100.50	100.51	100.48	100.51	100.51
18	562 700	536 550	546 700	535 750	629 040	548 180	547 290
19	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622
20	302.4	288.3	293.0	288.0	284.8	294.5	294.0
21	25.44	24.39	24.83	24.37	24.16	24.66	24.62
22	1.1923	1.1835	1.1873	1.1839	1.1800	1.1878	1.1875
23	1.1893	1.1827	1.1844	1.1816	1.1785	1.1855	1.1853
24	1.1923	1.1833	1.1872	1.1833	1.1813	1.1871	1.1869
25	561 170*	550 920*	543 010	536 450	531 210	545 890	543 000
26	11	6	8	25	23	8	41
27	70	66	59	60	67	33	69
28	1 820.0	1 871.1	1 800.8	1 231.6	1 156.8	1 278.0	1 175.1
29	0.27310	0.25790	0.23079	0.24055	0.24061	0.31025	0.30172
30	0.27310	0.25795	0.23078	0.24057	0.24075	0.31088	0.30185
31	3	11	11	3	11	11	11
32	500.46	500.55	500.50	500.58	500.65	500.66	500.55
33	11	11	11	3	11	11	11
34	500.63	500.65	500.50	500.29	500.62	500.66	500.60
35	0.9968	0.9985	0.9985	0.9985	0.9975	0.9983	0.9983
36	0.9988	0.9983	0.9984	0.9985	0.9984	0.9985	0.9985
37	0.9972	0.9983	0.9983	0.9985	0.9979	0.9983	0.9983
38	0.9987	0.9986	0.9987	0.9985	0.9985	0.9985	0.9985
39	499.58	499.81	500.51	500.29	500.18	500.65	500.55
40	12.981	12.992	13.003	12.730	12.971	13.050	12.625
41	25.985	25.994	25.976	25.410	25.933	25.074	25.225
42	0.28	0.28	0.28	0.28	0.28	0.28	0.28
43	25.705	25.714	25.696	25.130	25.653	25.794	24.945
44	499.90	499.76	500.49	500.64	500.52	500.50	500.56
45	56.866	60.862	55.124	54.004	53.582	50.064	49.051
46	113.925	121.880	110.226	108.124	107.050	100.028	97.984
47	0.9745	0.9762	0.9754	0.9762	0.9758	0.9753	0.9755
48	111.003	118.979	107.500	105.551	104.575	97.577	95.583
49	564 059	530 840	547 992	530 544	531 105	548 792	547 904
50	88.220	96.166	84.531	82.984	81.466	74.234	73.089
51	6 398.76	5 582.02	6 477.61	6 464.78	6 524.15	7 392.73	7 501.53
52	1 746.1	1 439.9	1 553.2	1 556.0	1 570.7	1 553.3	1 614.3
53	0.266	0.252	0.234	0.235	0.235	0.208	0.187
54	1 746.4*	1 440.2*	1 553.4	1 556.1	1 570.9	1 556.6	*1 614.4

Line	9	10	11	12	13	14
1	17.8	17.8	18.0	17.8	17.6	17.5
2	28.6	24.0	24.6	25.0	25.0	25.0
3	28.6	24.0	26.6	28.0	25.0	25.0
4	19.8	29.0	19.7	22.0	25.0	21.0
5	21.0	22.0	19.7	21.0	20.5	20.6
6	19.2	19.1	16.6	18.7	18.7	18.7
7	19.2	19.0	19.0	19.0	19.0	19.0
8	19.2	19.0	18.8	18.8	18.8	18.8
9	No. 18 791	No. 18 791	No. 18 791	No. 18 701	No. 18 791	No. 18 791
10	9.963	9.963	9.984	9.984	9.986	9.988
11	1 456, 1 476	1 456, 1 476	1 456, 1 470	1 456, 1 470	1 456, 1 476	1 456, 1 476
12	1 003.90	1 003.07	1 003.29	1 003.30	1 003.80	1 003.90
13	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047
14	10.017	10.017	10.018	10.018	10.015	10.015
15	54.083	54.106	53.448	55.556	50.841	50.792
16	5 399.10	5 407.40	5 335.20	5 545.80	5 075.00	5 070.10
17	100.60	100.60	100.49	100.49	100.47	100.47
18	543 610	543 440	536 180	558 280	509 900	509 [illegible]
19	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622
20	291.4	291.8	288.1	300.0	274.0	272.8
21	24.60	24.66	24.41	25.38	23.36	25.25
22	1.1862	1.1864	1.1830	1.1912	1.1758	1.1757
23	1.1830	1.1829	1.18[illegible]	1.1875	1.1730	1.1719
24	1.1852	1.1857	1.1835	1.1907	1.1761	1.1750
25	542 650	544 740	537 470	550 710	511 280	510 7[illegible]
26	9	16	10	16	8	44
27	55	66	61	67	87	72
28	1 658.5	1 794.8	1 832.3	1 801.8	1 089.4	1 070.7
29	0.23992	0.24056	0.23565	0.22033	0.22837	0.22140
30	0.24011	0.24070	0.23579	0.22040	0.22851	0.22155
31	11	98	3	11	3	3
32	500.55	500.82	500.40	500.89	500.43	500.41
33	11	28	3	11	3	3
34	500.61	500.82	500.40	500.61	500.41	500.41
35	0.9984	0.9977	0.9981	0.9977	0.9977	0.9980
36	0.9984	0.9985	0.9985	0.9985	0.9985	0.9985
37	0.9981	0.9978	0.9981	0.9981	0.9982	0.9982
38	0.9985	0.9985	0.9986	0.9986	0.9985	0.9986
39	500.50	499.92	500.35	500.23	500.28	500.18
40	13.066	18.179	13.275	13.320	13.366	13.275
41	26.101	26.3[illegible]2	26.537	26.679	26.571	26.542
42	0.28	0.28	0.28	0.28	0.28	0.28
43	25.821	26.082	26.257	26.399	26.294	26.262
44	500.41	499.97	500.50	500.86	500.21	500.31
45	57.650	54.325	51.495	51.515	53.425	52.481
46	115.231	109.135	102.945	103.908	106.655	104.900
47	0.9769	0.9768	0.9762	0.9748	0.9770	0.9779
48	112.478	106.494	100.495	100.605	104.435	102.583
49	543 588	534 634	537 370	550 090	511 116	510 [illegible]
50	89.430	83.103	76.691	77.659	80.530	78.6[illegible]
51	6 077.63	6 557.07	7 108.96	7 257.01	6 344.62	6 491.[illegible]
52	1 459.3	1 578.4	1 651.7	1 725.4	1 450.8	1 [illegible]
53	0.234	0.236	0.230	0.235	0.241	0.2[illegible]
54	1 459.5	*1 578.6	*1 651.9	*1 725.6	*1 451.0	1 [illegible]

* Note.—See table below

<——Long Marks (1st Calibration) on Gauge Board of Salt Solution Tank Used on These Tests——> <——Short Marks (2d Calibration) on Gauge Board of Salt Solution Tank Used on These Tests.——

<——Sampling Pipes on Lower Section.——>

NOTE.—Missing Tests, *A*, *D*, *E*, represent rough preliminary tests by chemicals, and are not included in this table. All other chemical tests appear in the tables in the order in which they were executed; missing numbers, or letters, relate to tests by current meters or to tests for purposes other than turbine performance.

Missing Tests, *B*, *C*, *F*, *J*, *P*, *Q*, *R*, 1-5, incl., represent current-meter tests.

Missing Test, *M*, represents a distribution test.

No Leeds and Northrup or Allis-Chalmers power readings taken on lettered tests. (High precision instruments.)

Some of the discharges are more or less questionable owing to the probability of leaks in the tail-race sampling pipes. In compiling this table, titrations of doubtful samples have been omitted, figures being substituted instead, which are based on the titrations of adjacent samples. The table on the right gives the discharge of Tests 8, 10-13 incl., using the actual titrations, no alterations, whatever, being made. These are the only tests thus treated as being doubtful, in this series.

*In Tests *T* and *U* the salt solution samples were undoubtedly interchanged by mistake in the laboratory. More correct values for the discharge in these tests can be computed by interchanging the values given above for v and recomputing Q_2. Thus, for Test *T*, Q_2 = 1 662, and for Test *U*, Q_2 = 1 518.

TABLE GIVING DISCHARGE OF TESTS 8, 10-13, INCL., USING ACTUAL DOSED TAIL-WATER TITRATIONS.

Line.	Formula.	Symbol.	Tests. 8	10	11	12	13
45	Titration of D. T. W		48.858	54.323	51.263	51.498	53.686
46	$\frac{\text{Line } 45 \times \text{Line } 38}{\text{Line } 44 \times \text{Line } 37}$	v_2	97.686	106.729	102.587	103.901	107.374
47		k	0.9755	0.9778	0.9762	0.9748	0.9779
48	Line 46 × Line 47	kv_2	95.246	106.006	100.097	100.260	104.903
49	Line 25 − Line 48	$v - kv_2$	547 906	544 634	537 370	550 600	511 115
50	Line 46 − Line 43	$v_2 - v_1$	72.893	82.647	76.330	77.412	81.080
51	Line 49 ÷ Line 50	r	7 587.25	6 589.96	7 014.70	7 221.02	6 311.1
52	Line 30 × Line 51	$Q_1 \; qr$	1 571.39	1 586.16	1 660.58	1 730.87	1 423.13
53	Line 30 × Line 47	kq	0.167	0.226	0.230	0.233	0.221
54	Line 52 + Line 53	Q_2	1 581.6	1 586.4	1 660.8	1 731.1	1 423.35

Number of line in this table corresponds to number of line in main table.

Test 202, September 30th, 1914.

Titration, October 2d.
Nitrate (1.56 grammes), September 9th.

Dilute salt solution. Temp. 19° cent. Flasks Nos. 1 456–1 474.		Normal head-water. Temp. 16.5° cent. Flasks Nos. 3–11.	Special dilutions.* Temp. 16.5° cent. Flasks Nos. 1 456– .		
5 050	5 050	1 295	5 000	5 065	10 065
5 052	5 058	1 289	4 995	5 053	10 048
5 033	5 040	1 283	5 035	5 087	10 122
5 070	5 070	1 283	5 085	5 055	10 140
			5 060	5 015	10 075
Mean = 50.529		Mean = 12.875	Mean = 50.450		

Tail-water Titrations.
Temp. 16.5° cent. Flasks Nos. 3-11.

5 235		5 185		5 170	
	5 185		5 205		5 145
5 250		5 265		5 310	
	5 468		5 800		5 794
5 582		†5 811		5 933	
	5 724		5 910		5 962

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	52 240	52 075	52.167
Lower.....	56 462	58 798	57.760
Averages.	54.351	55.437	54.963 = Arith. mean.

Weighted mean = 54.564

* 10 c.c. pipettes of salt solution from each of Samples ———, 30 c.c., diluted with 2 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† Average of two values to the right and the lower one to the left. Sample pipe was broken.

Test 203, October 1st, 1914.

Titration, October 3d.
Nitrate (1.56 grammes), September 9th.

Dilute salt solution. Temp. 17° cent. Flasks Nos. 1 456–1 476.		Normal head-water. Temp. 15° cent. Flask No. 11.	Special dilutions.* Temp. 15° cent. Flasks Nos. 1 456–1 476.		
5 110	5 115	1 270	038	5 053	10 091
5 115	5 120	1 282	5 035	5 060	10 095
5 112	5 118	1 280	5 040	5 050	10 090
5 136	5 140	1 280	5 020	5 073	10 093
			5 041	5 056	10 097
Mean = 51.208		Mean = 12.780	Mean = 50.466		

Tail-water Titrations.
Temp. 15.4° cent. Flask No. 11.

4 910		4 880		4 785	
	4 925		4 865		4 945
4 965		4 915		4 938	
	5 327		5 961		6 315
5 460		†5 905		6 343	
	5 668		6 085		6 376

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	49 190	48 842	49 031
Lower.....	55 900	62 160	59 378
Averages.	52.545	55.501	54.204 = Arith. mean.

Weighted mean = 53.646

* 30 c.c. salt solution from a mixture of Samples *A*, *C*, *E*, *G*, *D*, *K*, 30 c.c., diluted with 2 000 c.c. normal head-water and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† Average of the two values on the right and the lower one on the left. Sample pipe was broken.

*Test 204, October 2d, 1914.

Titration, October 4th.
Nitrate (1.56 grammes), September 9th.

Dilute salt solution. Temp. 17.75° cent. Flasks Nos. 1 456–1 474.		Normal head-water Temp. 16.3° cent. Flask No. 11.	Special dilutions.* Temp. 16.3° cent. Flasks Nos. 1 456–1 474.		
4 721	4 724	1 260	4 759	4 706	9 465
4 723	4 723	1 250	4 755	4 713	9 468
4 715	4 715	1 251	4 764	4 715	9 479
4 720	4 715	1 250	4 760	4 700	9 460
			4 750	4 710	9 460
Mean = 47.195		Mean = 12.528	Mean = 47.332		

Tail-water Titrations.
Temp. 16° cent. Flask No. 11.

4 581		4 609		4 580	
	4 607		4 625		4 652
4 650		4 630		4 719	
	5 105		5 633		5 825
5 145		†5 608		5 913	
	5 370		5 822		5 967

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	46 154	46 440	46.281
Lower.....	53 070	58 320	55.987
Averages.	49.612	52.380	51.134 = Arith. mean.

Weighted mean = 50.351.

*30 c.c. salt solution from a mixture of samples *A*, *C*, *E*, *G*, *D*. *K*, 30 c.c., diluted with 2 000 c.c, normal head-water, and 10 c.c. of this diluted to 1 000 c. c. with normal head-water. This liter then divided into two nearly equal parts. 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† Average of two values on the right and the lower one on the left. Sample pipe broken.

Test 205, October 5th, 1914.

Titration October 6th.
Nitrate (1.56 grammes), September 9th

Dilute salt solution. Temp. 23.5° cent. Flasks Nos. 1 456–1 474.		Normal head-water. Temp. 17° cent. Flask No. 11.	Special dilutions.* Temp. 17° cent. Flasks Nos. 1 456–1 474.		
4 653	4 644	1 240	5 082	5 043	10 125
4 642	4 644	1 251	5 065	5 053	10 118
4 635	4 645	1 255	5 072	5 030	10 102
4 645	4 645	1 239	5 050	5 035	10 085
			5 050	5 035	10 085
Mean = 46.441		Mean = 12.463	Mean = 50.515		

Tail-water Titrations.
Temp. 16° cent. Flasks Nos. 3–11.

5 647		5 703		5 450	
	5 687		5 560		5 430
5 710		5 635		5 507	
	5 255		4 000		3 982
4 753		3 895		3 755	
	4 214		3 680		3 750

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	56 764	54 868	55.921
Lower.....	45 292	38 334	41.427
Averages.	51.028	46.601	48.674 = Arith. mean.

Weighted mean = 49.797.

*10 c.c. salt solution from each of Samples *B*, *D*. *F*, *I*, *K*. 50 c.c.. diluted with 3 000 c c. normal head-water, and 10 c.c of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c in a casserole for titration.

TABLE 51.—DATA FOR COMPUTATION OF TURBINE DISCHARGE. COMPILED FROM RECORDS OF TESTS MADE ON UNIT NO. 13, AUGUST 19TH TO SEPTEMBER 2D, 1914.—SEE NOTE BELOW.

Line.	UNIT No. 13.	Formula.	Symbol.	Units.
1	Temperature: Salt solution in tank during test			°C
2	Temperature: Salt solution when pipetted for dilution with distilled water			°C
3	Temperature: Distilled water for dilution of salt solution			°C
4	Temperature: Normal head water when measured for evaporation in laboratory			°C
5	Temperature: Dosed tail water " " " " "			°C
6	Temperature: Normal head-water in canal			°C
7	Temperature: Dosed tail water in tail race			°C
8	Mean temperature of head and tail water during test, mean of Line 6 and Line 7			°C
9	Number of pipette used in measuring out salt solution for dilute salt solution at temperature of Line 2			
10	Volume of salt solution discharged by pipette for dilute salt solution at temperature of Line 2			c.c.
11	Numbers of flasks used in measuring distilled water			
12	Volume of flask measuring distilled water for dilution of salt solution at temperature of Line 3			c.c.
13	Number of pipette used in measuring out dilute salt solution			
14	Volume of dilute salt solution discharged by pipette at temperature of Line 3			c.c.
15	Titration per pipette of dilute salt solution in laboratory			c.c.
16	Titration per liter of dilute salt solution in laboratory at temperature of Line 3	Line 15 ÷ Line 14		c.c.
17	Ratio of dilution of salt solution with distilled water	Line 12 ÷ Line 10		
18	Titration per liter of salt solution at temperature of Line 2	Line 16 × Line 17		c.c.
19	Concentration of silver nitrate, grammes per liter (approximate)			gms.
20	Concentration (approximate) in grammes per liter of salt solution at temperature of Line 2			gms.
21	Percentage of salt in solution (approximate) at temperature of Line 2	$p = \frac{c}{d}$ (from curves)		
22	Density: Salt solution in tank during test at temperature of Line 1			
23	Density: " " when pipetted for dilution with distilled water at temperature of Line 2			
24	Density: " " at mean temperature of head and tail-water at temperature of Line 8			
25	Titration of salt solution per liter at mean temperature of head and tail water at temperature of Line 8	Line 18 × Line 24 ÷ Line 23	v	c.c.
26	Tank gauge board reading at start			div.
27	" " " " finish			div.
28	Time, in seconds, from first to last gauge reading			sec.
29	Actual rate of discharge of salt solution from tank during test at temperature of Line 1	$\frac{8\ 466 \times (\text{Line 27} - \text{Line 26})}{\text{Line 28}}$		cu. ft.
30	Equivalent rate of discharging salt solution at mean temperature of head and tail-water, temperature of Line 8	Line 29 × Line 22 ÷ Line 24	q	cu. ft.
31	Numbers of flasks used in measuring normal head-water samples			
32	Average volume of flasks measuring normal head water samples at temperature of Line 4			c.c.
33	Numbers of flasks used in measuring dosed tail water samples			
34	Average volume of flasks measuring dosed tail water samples, at temperature of Line 5			c.c.
35	Density: Normal head-water when measured in laboratory, at temperature of Line 4			
36	Density: Normal head water at mean temperature of head and tail-water, temperature of Line 8			
37	Density: Dosed tail water when measured in laboratory at temperature of Line 5			
38	Density: Dosed tail-water at mean temperature of head and tail water, temperature of Line 8			
39	Volume of normal head-water samples at mean temperature of head and tail-water, temperature of Line 8	Line 32 × Line 35 ÷ Line 36		c.c.
40	Titration of normal head-water samples in laboratory			c.c.
41	Titration per liter of normal head-water samples at mean temperature of head and tail-water, temperature of Line 8	Line 40 ÷ Line 39		c.c.
42	Correction to titration of normal head-water per liter			c.c.
43	Correct titration per liter of normal head water at mean temperature of head and tail water, temperature of Line 8	Line 41 — Line 42	v_1	c.c.
44	Volume of dosed tail-water samples at mean temperature of head and tail-water, temperature of Line 8	Line 34 × Line 37 ÷ Line 38		c.c.
45	Titration of dosed tail-water samples at temperature of tail-water when measured for evaporation, temperature of Line 5			c.c.
46	Titration per liter of dosed tail-water samples at mean temperature of head and tail-water, temperature of Line 8	$\frac{\text{Line 45} \times \text{Line 38}}{\text{Line 44} \times \text{Line 37}}$	v_2	c.c.
47	Approximate value of k		k	
48	Value of $k\,v_2$	Line 46 × Line 47	$k\,v_2$	
49	Value of $v - k\,v_2$	Line 25 — Line 48	$v - k\,v_2$	
50	Value of $v_2 - v_1$	Line 46 — Line 43	$v_2 - v_1$	
51	Value of r	Line 49 ÷ Line 50	r	
52	Value of $Q_1 = q\,r$	Line 30 × Line 51	Q_1	
53	Value of $k\,q$	Line 30 × Line 47	$k\,q$	
54	Value of $Q_2 = Q_1 + k\,q$	Line 52 + Line 53	Q_2	

←Sampling pipes on lower section and also eight samples on upper section in Tests 19, 25, and 23.→ ←Sampling pipes on upper section.→

←Short marks (second calibration) on gauge-board of salt solution tank used on these tests.→

NOTE.—See Table 52 for Tests 29 B and 35 B. * See table below for discharge. † Test 27 retitrated as Test 27 B 18 days after test of turbine. ‡ Test 22 retitrated as Test 26 B 15 days after the turbine test. There is, therefore, some doubt as to the accuracy of 26 B. It would be better, perhaps, to disregard the B tests entirely.

Missing Tests 24, 25, 31–31, incl., and 34–36, incl., represent current-meter tests. Some of the discharges, Tests 15–21, incl., are more or less questionable, owing to the probability of leaks in tail race sampling pipes. In compiling the above table titrations of doubtful samples have been omitted, figures being substituted, which are based on the titrations of adjacent samples. The table on the left gives the discharge of Tests 15, 17–21, incl., using the actual titrations, no alterations, whatever, being made.

§ The titrations of tail water samples in Test 45 B were made about a week after Test 45. They indicate that the samples had become contaminated, one of them being 72.45 c.c. as against about 54 c.c., the correct value. This reduction of Test 45, consequently, is disregarded.

TABLE GIVING DISCHARGE OF TESTS 15, 17–21, INCL., USING ACTUAL DOSED TAIL-WATER TITRATIONS.

Line.	Formula.	Symbol.	Tests 15	17	18	19	20	21
45	Titration D. T. W.	....	46 560	41.567	47 654	56 103	60.073	42 610
46	$\frac{\text{Line 45} \times \text{Line 38}}{\text{Line 44} \times \text{Line 37}}$	v_2	99 660	85.680	96 665	116.364	106.188	66 798
47	....	k	0.9766	0.9759	0.9708	0.9747	0.9764	0.9764
48	Line 46 × Line 47	$k\,v_2$	97 067	83 622	92 731	113 636	97.798	97.443
49	Line 25 — Line 48	$v - k\,v_2$	584 463	618 587	643 256	504 597	536 323	533 885
50	Line 46 — Line 43	$v_2 - v_1$	79.641	60.580	70 600	91 220	74.232	74 425
51	Line 49 ÷ Line 50	r	7 336 30	7 611.64	7 795.09	6 183 61	7 160.16	7 166.25
52	Line 30 × Line 51	$Q_1 = q\,r$	1 855 00	1 649 05	1 635 82	1 617.21	1 504 55	1 589.04
53	Line 30 × Line 47	$k\,q$	0.210	0.200	0.207	0 239	0.217	0 217
54	Line 52 + Line 53	Q_2	1 855.2	1 660 8	1 635.0	1 617.4	1 591.8	1 589.3

Number of line in this table corresponds to number of line in main table.

Test 206, October 6th, 1914.

Titration, October 7th.
Nitrate (1.56 grammes), October 5th.

Dilute salt solution. Temp. 18° cent. Flasks Nos. 1 456-1 476.		Normal head-water. Temp. 13.8° cent. Flask No. 11.	Special dilutions.* Temp. 13.8° cent. Flasks Nos. 1456-1476.		
4 550	4 560	1 253	4 965	4 960	9 925
4 560	4 552	1 262	5 005	4 970	9 975
4 561	4 552	1 255	4 985	4 983	9 968
4 558	4 559	1 256	4 990	4 978	9 968
			4 948	5 018	9 966
Mean = 45.565		Mean = 12.565	Mean = 49.802		

Tail-Water Titrations.
Temp. 14° cent. Flasks Nos. 3-11.

5 385		5 330		5 043	
	5 340		5 146		4 863
5 405		5 233		5 030	
	5 247		4 370		4 510
4 580		4 460		4 504	
	4 525		4 488		4 478

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper....	53 386	50 205	51.972
Lower....	47 030	44 700	45.736
Averages.	50.208	47.452	48.854 = Arith. mean.

Weighted mean = 49.480

*10 c.c. salt solution from each of Samples *B, D, F, I. K*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each. for evaporation to about 10 c.c. in a casserole for titration.

Test 207, October 7th, 1914.

Titration, October 8th.
Nitrate (1.56 grammes), October 5th.

Dilute salt solution, Temp. 18.5° cent. Flasks Nos. 1 456-1 476.		Normal head-water, Temp. 12.75° cent. Flask No. 11.	Special dilutions.* Temp. 12.75° cent. Flasks Nos. 1 456-1 476.		
4 684	4 679	1 253	5 119	5 032	10 151
4 682	4 680	1 250	5 107	5 053	10 160
4 680	4 680	1 252	5 119	5 040	10 159
4 675	4 680	1 255	5 080	5 065	10 145
			5 075	5 070	10 145
Mean = 46.800		Mean = 12.525	Mean = 50.760		

Tail-Water Titrations.
Temp. 14° cent. Flasks Nos. 3-11.

5 515		5 447		5 075	
	5 500		5 275		4 950
5 560		5 325		5 197	
	5 330		4 718		4 749
4 620		4 690		4 651	
	4 650		4 850		4 726

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper....	54 694	51 242	53.160
Lower....	48 225	47 388	47.760
Averages.	51.460	49.315	50.460 = Arith. mean.

Weighted mean = 50.685

*10 c.c. salt solution from each of Samples *B, D, F, I, K*. 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c. c. of this diluted to 1000 c. c. with normal head-water. This liter then divided into two nearly equal parts, 500 c. c. each, for evaporation to about 10 c.c. in a casserole for titration.

Test 208, October 8th, 1914.

Titration, October 9th.
Nitrate (1.56 grammes), October 5th.

Dilute salt solution. Temp. 18.25° cent. Flasks Nos. 1 456–1 476.		Normal head-water. Temp. 16.7° cent. Flask No. 11.	Special dilutions.* Temp. 18° cent. Flasks Nos. 1 456–1 476.		
4 830	4 840	1 250	5 250	5 202	10 452
4 835	4 830	1 258	5 312	5 210	10 522
4 845	4 838	1 260	5 255	5 205	10 460
4 838	4 837	1 261	5 245	5 180	10 425
			5 210	5 200	10 410
Mean = 48.366		Mean = 12.572	Mean = 52.269		

Tail-Water Titrations.
Temp. 16.25° cent. Flask No. 11.

5 750		5 708		5 163	
	5 655		5 432		5 025
5 722		5 520		5 260	
	5 509		4 585		4 540
4 800		4 615		4 569	
	4 670		4 556		4 501

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper	56 710	52 200	54.706
Lower	48 985	45 502	47.050
Averages.	52.848	48.851	50.878 = Arith. mean.

Weighted mean = 51.753

* 10 c.c. salt solution from each of Samples *B*, *D*, *F*, *I*, *K*, 50 c.c., diluted with 3 000 c.c. normal head water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

Test 209, October 9th, 1914.

Titration, October 10th.
Nitrate (1.56 grammes), October 5th.

Dilute salt solution. Temp. 21° cent. Flasks Nos. 1 456–1 476.		Normal head-water. Temp. 16° cent. Flasks Nos. 3–11.	Special dilutions.* Temp. 16° cent. Flasks Nos. 1 456–1 476.		
4 990	4 990	1 252	5 370	5 295	10 665
4 987	4 990	1 264	5 375	5 295	10 670
4 980	4 990	1 252	5 350	5 345	10 695
5 002	5 003	1 260	5 370	5 305	10 675
			5 360	5 315	10 675
Mean = 49.915		Mean = 12.570	Mean = 53.380		

Tail-Water Titrations.
Temp. 16° cent. Flasks Nos. 3–11.

5 752		5 800		5 020	
	5 602		5 365		4 820
5 615		5 450		5 206	
	5 225		4 772		4 542
4 850		4 782		4 550	
	4 815		4 610		4 485

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper	56 438	51 028	54.033
Lower	49 180	45 918	47.368
Averages.	52.809	48.473	50.701 = Arith. mean.

Weighted mean = 51.477

* 10 c.c. salt solution from each of Samples *B*, *D*, *F*, *I*, *K*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

TABLE 52.—DATA FOR COMPUTATION OF TURBINE DISCHARGE. COMPILED FROM RECORDS OF TESTS MADE ON UNITS NOS. 11, 12, AND 13. TESTS 101–104, INCLUSIVE, ARE DISTRIBUTION TESTS.

Column groups: UNIT No. 11.—DOSED TAIL SAMPLING PIPES ON UPPER SECTION, SEPT. 12 TO SEPT. 18, 1914. THIS UNIT CONTINUED ON TABLE 53. (Tests 106, 100, 181, 108, 109). UNIT No. 12.—DOSED TAIL SAMPLING PIPES ON UPPER SECTION. SEPT. 26 TO OCT. 13, 1914. (Tests 200 to 212). UNIT No. 13. SEE TABLES 56 AND 61. (Tests 23 B, 26 B).

Line		Formula	Symbol	Units
1	Temperature. Salt solution in tank during test			°C
2	Temperature. Salt solution when pipetted for dilution with distilled water			°C
3	Temperature. Salt solution when pipetted for special dilutions			°C
4	Temperature. Distilled water for dilution of salt solution			°C
5	Temperature. Normal head-water for special dilutions			°C
6	Temperature. Normal head-water when measured for evaporation in laboratory			°C
7	Temperature. Dosed tail-water when measured for evaporation in laboratory			°C
8	Temperature. Dosed tail-water in tail-race			°C
9	Temperature. Normal head-water in canal			°C
10	Mean temperature of head and tail-water during test, mean of Line 8 and Line 9	(Line 8 + Line 9) ÷ 2		°C
11	Pipette used in measuring out salt solution for dilute salt solution at temperature of Line 2			
12	Volume of salt solution discharged by pipette for dilute salt solution at temperature of Line 2			c.c.
13	Flask used in measuring distilled water to dilute salt solution			
14	Average volume of flasks measuring distilled water for dilution of salt solution at temperature of Line 4			c.c.
15	Pipette used in measuring out dilute salt solution			
16	Volume of dilute salt solution discharged by pipette at temperature of Line 4			c.c.
17	Titration per pipette of dilute salt solution in laboratory			c.c.
18	Titration per liter of dilute salt solution in laboratory	Line 17 ÷ Line 16		c.c.
19	Ratio of dilution of salt solution with distilled water	Line 14 ÷ Line 12		
20	Titration per liter of salt solution at temperature of Line 2	Line 18 × Line 19		c.c.
21	Concentration of silver nitrate, in grammes per liter (approximate)			
22	Concentration (approximate), in grammes per liter of salt solution at temperature of Line 2			
23	Percentage of salt in solution at temperature of Line 2	$p = c \div d$ (from curve)		
24	Density. Salt solution in tank during test at temperature of Line 1			
25	Density. Salt solution when pipetted for dilution with distilled water at temperature of Line 2			
26	Density. Salt solution when pipetted for stock solutions at temperature of Line 3			
27	Density. Salt solution at mean temperature of head and tail-water at temperature of Line 10			
28	Titration per liter of salt solution in tank during test at temperature of Line 1	Line 20 × Line 24 ÷ Line 25		c.c.
29	Titration per liter of salt solution in tank at mean temperature of head and tail-water, temperature of Line 10	Line 20 × Line 27 ÷ Line 25	v	c.c.
30	Tank gauge-board reading at start			Div.
31	Tank gauge-board reading at finish			Div.
32	Time, in seconds, from first to last gauge reading			sec.
33	Actual rate of discharge of salt solution from tank at temperature of Line 1	[illegible] × (Line 31 − Line 30) ÷ Line 32		cu. ft.
34	Equivalent rate of discharging salt solution at mean temperatures of head and tail-water, temperature of Line 10	Line 33 × Line 24 ÷ Line 27	q	cu. ft.
35	Volume of salt solution discharged by pipette for stock solutions, temperature of Line 3			c.c.
36	Volume of salt solution discharged by pipette for stock solutions at mean temperature of head and tail-water, temperature of Line 10	Line 35 × Line 26 ÷ Line 27		c.c.
37	Flasks used for special dilutions			
38	Nominal volume of normal head water for stock solutions = V. No. of 10 c.c. pipettes of salt solution = m			
39	Volume of normal head water for stock solutions at temperature of Line 5			c.c.
40	Density of normal head water for special dilutions at temperature of Line 5			
41	Density of normal head-water when measured in laboratory at temperature of Line 6			
42	Density of normal head-water at mean temperature of head and tail-water, temperature of Line 10			
43	Volume of normal head-water for special dilutions at mean temperature of head and tail-water, temperature of Line 10	Line 39 × Line 40 ÷ Line 42		c.c.
44	Ratio of mixture, normal head-water to salt solution for stock bottle, mean temperature of head and tail-water, temperature of Line 10	Line 43 ÷ m × Line 36	r_a	
45	Value of k_a		k_a	
46	Ratio of dilution of normal head-water and salt solution for stock bottles, mean temperature of head and tail-water, temperature of Line 10	Line 44 + Line 45	R_a	
47	Volume discharged by pipette measuring stock solution, temperature of Line 5			c.c.
48	Density of stock solution, temperature of Line 5			
49	Density of stock solution, mean temperature of head and tail-water, temperature of Line 10			
50	Volume of stock solution when pipetted at mean temperature of head and tail water, temperature of Line 10	Line 47 × Line 48 ÷ Line 49		c.c.
51	Volume of flask measuring normal head water for dilution of stock solutions, temperature of Line 5			c.c.
52	Volume of normal head-water for dilution of stock solutions at mean temperature of head and tail-water, temperature of Line 10	Line 51 × Line 40 ÷ Line 42		c.c.
53	Ratio of dilution of normal head water with stock solution at mean temperature of head and tail-water, temperature of Line 10	Line 52 ÷ Line 50	R_b	
54	Ratio of dilution for special dilutions at mean temperature of head and tail-water, temperature of Line 10	Line 53 × Line 46	R'	
55	Value of k'		k'	
56	Value of $r' = R' - k'$	$R' - k'$	r'	
57	Value of k taken same as k' except where v_2 and v'_2 differ widely		k	
58	Flasks measuring special dilution			
59	Average volumes of special dilutions for evaporation, temperature of Line 5. (½-liter flask vol. evaporated)			c.c.
60	Titration per sample of special dilutions evaporated			c.c.
61	Titration per liter of special dilutions evaporated at temperature of Line 5	Line 60 ÷ Line 59		c.c.
62	Density of special dilutions at temperature of Line 5			
63	Density of special dilutions at mean temperature of head and tail-water, temperature of Line 10			
64	Titration per liter of special dilutions at mean temperature of head and tail-water, temperature of Line 10	Line 61 × Line 63 ÷ Line 62	v'_2	c.c.
65	Flasks used in measuring normal head-water samples			
66	Average volume of flasks measuring normal head-water samples at temperature of Line 6			c.c.
67	Flasks used in measuring dosed tail-water samples			
68	Average volume of flasks measuring dosed-tail-water samples at temperature of Line 7			c.c.
71	Density. Dosed tail water at temperature of Line 7			
72	Density. Dosed tail-water at mean temperature of head and tail water, temperature of Line 10			
73	Volume of normal head-water samples at mean temperature of head and tail-water, temperature of Line 10	Line 66 × Line 41 ÷ Line 42		c.c.
74	Titration of normal head-water samples in laboratory			c.c.
75	Titration per liter of normal head-water at mean temperature of head and tail-water, temperature of Line 10	Line 74 ÷ Line 73		c.c.
76	Correction per liter to titration of normal head water			c.c.
77	Correct titration per liter of normal head-water at mean temperature of head and tail-water, temperature of Line 10	Line 75 − Line 76	v_1	c.c.
78	Volume of dosed tail water samples at mean temperature of head and tail-water, temperature of Line 10	Line 68 × Line 71 ÷ Line 72		c.c.
79	Titration per sample of dosed tail water in laboratory, temperature of Line 7			c.c.
80	Titration per liter of dosed tail-water, at temperature of Line 7	Line 79 ÷ Line 68		c.c.
81	Titration per liter of dosed tail-water at mean temperature of head and tail-water, temperature of Line 10	Line 80 × Line 72 ÷ Line 71	v_2	c.c.
82	Value of $v_2 - v'_2$	Line 81 − Line 64	$v_2 - v'_2$	
83	Value of $k' v_2$ (k' used in place of k''. See Equations (98) and (105))	Line 55 × Line 81	$k' v_2$	
84	Value of $v - k' v_2$	Line 29 − Line 83	$v - k' v_2$	
85	Value of $R' \times (v_2 - v'_2)$	Line 54 × Line 82		
86	Value of $R' \times (v_2 - v'_2) \div (v - R' v_2) = f$	Line 85 ÷ Line 84	f	
87	Value of $1 + f$	Line 86 + 1	$1 + f$	
88	Value of $r = r' + (1 + f)$	Line 56 ÷ Line 87	r	
89	Value of $Q = q\,r$	Line 34 × Line 88	Q	
91	Value of k	Same as Line 57	k	
92	Value of $k\,q$	Line 34 × Line 91	$k\,q$	
93	Value of $Q_1 = Q + k\,q$	Line 90 + Line 92	Q_1	
94	Value of k	Same as Line 57	k	
95	Value of $k\,v_2$	Line 94 × Line 81	$k\,v_2$	
96	Value of $v - k\,v_2$	Line 29 − Line 95	$v - k\,v_2$	
97	Value of $v_2 - v_1$	Line 81 − Line 77	$v_2 - v_1$	
98	Value of r	Line 96 ÷ Line 97	r	
99	Value of $Q_1 = q\,r$	Line 98 × Line 34	Q_1	
100	Value of $k\,q$	Line 94 × Line 34	$k\,q$	
101	Value of $Q_2 = Q_1 + k\,q$	Line 99 + Line 100	Q_2	
102	Average value of Q_2	(Line 93 + Line 101) ÷ 2		

*0.03 c.c. $AgNO_3$ has been deducted from titration of salt soln. (Test 107) upon application of Table 8.

† Value obtained by computing R' as R' (Line 54, Test 107) appears to be in error.

$R' = \frac{v - k\,v_2}{v_2 - v_1}$ R' by formula = 7 771.90.

* Test 23 retitrated as Test 23 B one month after the test and 26 B 10 days after the test. Tests 23 B and 26 B are disregarded in the final computations owing to the probability of changes in samples when stored for so long a time.

Test 210, October 10th, 1914.

Titration, October 12th.
Nitrate (1.56 grammes), October 5th.

Dilute salt solution. Temp. 18° cent. Flasks Nos. 1 456-1 476.		Normal head-water. Temp. 17.6° cent. Flask No. 11.	Special dilutions.* Temp. 17.6° cent. Flasks Nos. 1 456-1 476.		
4 834	4 823	1 257	5 195	5 186	10 381
4 827	4 820	1 250	5 166	5 197	10 363
4 825	4 830	1 260	5 197	5 200	10 397
4 825	4 825	1 250	5 215	5 175	10 390
			5 185	5 190	10 375
Mean = 48.261		Mean = 12.542	Mean = 51.906		

Tail-Water Titrations.
Temp. 17.6° cent. Flask No. 11.

5 150		5 227		4 851	
	5 060		4 920		4 777
5 028		4 932		4 860	
	5 100		5 015		5 005
5 185		5 145		5 093	
	5 150		5 125		5 020

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	50 794	48 520	49.783
Lower.....	51 450	50 516	50.931
Averages.	51.122	49.518	50.357 = Arith. mean.

Weighted mean = 50.664

*10 c.c salt solution from each of Samples *B*, *D*. *F*, *I*, *K*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

Test 211, October 12th, 1914.

Titration. October 13th.
Nitrate (1.56 grammes), October 12th.

Dilute salt solution. Temp. 18.25° cent. Flasks Nos. 1 456-1 476.		Normal head-water. Temp. 13.8° cent. Flask No. 3.	Special dilutions.* Temp. 13.8° cent. Flasks Nos. 1 456-1 476.		
4 855	4 870	1 255	5 305	5 205	10 510
4 860	4 860	1 255	5 245	5 210	10 455
4 858	4 862	1 253	5 190	5 231	10 421
4 877	4 870	1 255	5 234	5 218	10 452
			5 248	5 235	10 483
Mean = 48.640		Mean = 12.545	Mean = 52.321		

Tail-Water Titrations.
Temp. 16° cent. Flask No. 3.

5 160		5 165		4 848	
	5 140		4 915		4 787
5 111		4 974		4 850	
	5 038		5 080		5 060
5 180		5 070		5 109	
	5 070		5 111		5 100

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages
Upper.....	51 100	48 500	49.944
Lower.....	50 895	50 920	50.909
Averages	50.997	49.710	50.427 = Arith. mean.

Weighted meau = 50.636

*10 c.c. salt solution from each of Samples *B*, *D*, *F*, *I*, *K*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

Test 212, October 13th, 1914.

Titration, October 14th.

Nitrate (1.53 grammes), October 13th.

Dilute salt solution. Temp. 15.5° cent. Flasks Nos. 1 456–1 476.		Normal head-water. Temp. 11° cent. Flask No. 3.	Special dilutions.* Temp. 11° cent. Flasks Nos. 1 456–1 476.		
4 870	4 875	1 255	5 254	5 198	10 452
4 875	4 875	1 255	5 259	5 240	10 499
4 882	4 882		5 234	5 235	10 469
4 878	4 884		5 245	5 230	10 475
			5 245	5 215	10 460
Mean = 48.776		Mean = 12.550	Mean = 52.355		

* 10 c.c. salt solution from each of Samples *B*, *D*, *F*, *I*, *K*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

Tail-Water Titrations.

Temp. 13° cent. Flask No. 3.

5 477		5 402		5 165	
	5 476		5 205		5 100
5 490		5 230		5 109	
	5 025		5 177		5 217
5 052		5 177		5 245	
	5 050		5 240		5 235

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	54 150	51 447	52.949
Lower.....	50 760	52 228	51.576
Averages.	52.455	51.838	52 262 = Arith. mean.

Weighted mean = 52.274

or letters, relating to other than regular turbine tests. Such, for example, are Tests *M,* which consisted of a series relating to the distribution of salt in head- and tail-races.

The concentrations of salt solution, normal head-water, special dilution, and tail-water are, of course, the mean values computed from the titration tables, Sections 103, 104, 105, and 106, or from similar tables on file as explained in Section 102.

In the case of the salt-solution, normal head-water, or special dilution samples, the simple arithmetical mean has been used, sometimes cutting out titrations which were questionable. The particular titrations thus cut out are enclosed in parentheses in the tables of titrations.

In the case of the tail-water, the weighted mean for the average titrations of the four quarters of the draft-tube has been used, the weights being determined from Figs. 37, 38, and 39. These weights are merely the four relative mean velocities for the four quarters of the draft-tube, as determined by the Ott current meters, uncorrected for their deviations from the still-water ratings.

Where the normal head-water has been titrated directly, a correction for the error of such a titration, as explained in Section 43 *et seq.,* has been made. Where the normal head-water titration is computed from

TABLE 53.—DATA FOR COMPUTATION OF TURBINE DISCHARGE. COMPILED FROM RECORDS OF TESTS MADE ON UNIT NO. 11, NOV. 9TH TO NOV. 21ST, 1914. SEE TABLE 52 FOR FIRST SERIES OF TESTS ON UNIT NO. 11.

Line	UNIT No. 11.	Formula.	Symbol.	Units.	TEST 111	112	113	114	115	116	117	118	119	120 A	121 A	122 A	120 B	121 B	122 B
1	Temperature: Salt solution in tank during test			°C	5.7	6.0	4.8	4.5	4.8	4.0	4.0	4.2	2.5	2.2	2.8	2.0	2.2	2.8	2.0
2	Temperature: Salt solution pipetted for dilution with distilled water			°C	8.2	8.9	4.0	6.0	7.6	13.5	9.0	7.0	2.0	4.2	6.0	2.0	20.0	20.0	18.8
3	Temperature: Salt solution when pipetted for special dilutions			°C	5.8	8.0	8.2	4.0	5.4	6.0	9.0	7.0	1.2	4.0	4.6	2.0	4.0	4.5	2.0
4	Temperature: Distilled water for dilution of salt solution			°C	18.0	16.9	20.0	21.0	19.0	19.0	27.0	19.0	19.5	20.0	21.5	18.0	20.5	20.5	14.0
5	Temperature: Head-water for special dilutions			°C	8.5	7.3	9.8	7.5	8.6	5.1	9.0	7.7	4.6	6.0	7.5	7.0	6.0	12.75	8.2
6	Temperature: Dosed tail-water when measured in laboratory			°C	8.5	8.0	9.0	8.5	8.5	9.5	8.2	8.0	4.5	6.1	7.2	5.0	12.5	12.9	12.2
7	Temperature: Dosed tail-water in tail-race			°C	8.4	8.0	7.2	6.0	7.5	5.9	7.0	8.0	5.0	5.2	5.0	4.5	5.8	5.0	4.5
8	Temperature: Normal head-water in canal			°C	7.0	8.0	8.0	8.0	8.0	6.0	7.0	8.0	5.0	5.0	4.5	4.0	5.0	4.8	4.0
9	Mean temperature of head and tail-water during test, mean of Line 7 and Line 8	(Line 7 + Line 8) ÷ 2		°C	7.7	8.0	8.0	8.0	7.8	6.4	7.0	8.0	5.0	5.2	4.8	4.2	5.2	4.8	4.2
10	Numbers of pipettes measuring salt solution at temperature of Line 2			No.	9 171, 9 206	9 171, 9 206	9 171, 9 200	9 171, 9 206	9 171, 9 206	9 171, 9 206	9 171, 9 205	9 171, 9 206	9 171, 9 205	9 171, 9 206	9 171, 9 206	9 171, 9 206	9 171, 9 206	9 171, 9 200	9 171, 9 206
11	Volume of salt solution discharged by pipette at temperature of Line 2			c.c.	9.965	9.968	9.962	9.964	9.969	9.978	9.966	9.965	9.960	9.965	9.964	9.961	9.979	9.970	9.975
12	Flask numbers used in measuring distilled water to dilute salt solution			No.	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 476	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 487	1 407, 1 457
13	Volume of flask measuring distilled water for dilution of salt solution at temperature of Line 4			c.c.	1 001.25	1 001.27	1 001.3	1 001.32	1 001.27	1 001.27	1 001.68	1 001.27	1 001.29	1 001.2	1 001.34	1 001.25	1 001.31	1 001.21	1 001.15
14	Dilute salt solution discharged by pipette at temperature of Line 4			c.c.	10.004	10.003	10.006	10.007	10.006	10.006	10.007	10.006	10.006	10.006	10.008	10.004	10.007	10.007	10.000
15	Titration per pipette of dilute salt solution in laboratory			c.c.	59.944	61.575	59.695	62.663	54.944	58.714	53.625	62.141	61.528	59.423	59.685	53.153	58.988	59.965	53.150
16	Titration per liter of dilute salt solution in laboratory	Line 15 ÷ Line 14		c.c.	5 992.0	6 154.4	5 965.7	6 261.3	5 491.7	5 868.5	5 358.0	6 211.0	6 160.1	5 939.4	5 963.7	5 314.2	5 894.7	5 993.1	5 318.0
17	Ratio of dilution of salt solution with distilled water	Line 13 ÷ Line 11			100.48	100.49	100.51	100.40	100.44	100.40	100.51	100.48	100.53	100.48	100.50	100.49	100.34	100.34	100.30
18	Titration per liter of salt solution at temperature of Line 2	Line 16 × Line 17		c.c.	602 078	618 394	629 786	628 258	551 586	589 197	639 045	624 081	618 270	580 743	599 352	534 094	591 474	601 347	533 714
19	Concentration (approximate) grammes per liter salt solution at temperature of Line 2				301.64	309.20	314.86	314.63	275.73	294.50	319.52	312.09	306.14	298.37	299.66	267.01	295.74	300.30	266.86
20	Percentage of salt in solution at temperature of Line 2	$p = \frac{c}{d}$ (from curve)			26.20	26.75	26.15	26.13	23.40	24.73	26.60	25.95	26.72	24.60	25.10	22.70	24.90	25.26	22.75
21	Density: Salt solution in tank during test at temperature of Line 1				1.1965	1.2012	1.2048	1.2047	1.1818	1.1935	1.2062	1.2082	1.2024	1.1236	1.1963	1.1771	1.1954	1.1982	1.1778
22	Density: Salt solution when pipetted for dilution with distilled water at temperature of Line 2				1.1960	1.2005	1.2050	1.2042	1.1804	1.1894	1.2057	1.2021	1.2023	1.1930	1.1955	1.1771	1.1872	1.1900	1.1794
23	Density: Salt solution when pipetted for stock solutions at temperature of Line 3				1.1963	1.2000	1.2032	1.2049	1.1806	1.1928	1.2057	1.2021	1.2023	1.1922	1.1937	1.1771	1.1947	1.1974	1.1774
24	Density: Salt solution at mean temperature at head and tail-water at temperature of Line 9				1.1956	1.2000	1.2034	1.2034	1.1808	1.1925	1.2068	1.2025	1.2012	1.1916	1.1957	1.1762	1.1942	1.1974	1.1768
25	Titration per liter of salt solution in tank during test of temperature of Line 1	Line 18 × Line 21 ÷ Line 22		c.c.	602 257	518 766	629 640	629 510	552 248	591 900	640 385	624 648	618 206	597 312	599 712	531 024	595 565	605 395	536 169
26	Titration per liter of salt solution in tank at mean temperature of head and tail-water, temperature of Line 9	Line 18 × Line 24 ÷ Line 22	v	c.c.	601 895	516 147	628 947	628 813	551 531	590 729	639 516	624 268	617 714	596 687	599 412	533 597	594 984	604 975	535 742
27	Tank gauge board reading at start			div.	17	20	14	18	16	38	17	13	24	31	28	37	31	30	37
28	Tank gauge board reading at finish			div.	66	64	49	51	50	71	51	47	59	66	73	68	68	73	68
29	Time, in seconds, from first to last gauge reading			sec.	1 200.2	1 184.1	1 190.4	1 221.7	1 189.3	1 172.7	1 190.1	1 191.6	1 228.7	1 213.4	1 194.0	1 197.2	1 212.4	1 194.0	1 197.2
30	Actual rate of discharge of salt solution from tank at temperature of Line 1	$\frac{8.466 \times (\text{Line 28} - \text{Line 27})}{\text{Line 29}}$		cu. ft.	0.27469	0.24309	0.24705	0.24254	0.24208	0.23722	0.24085	0.24156	0.24116	0.24420	0.24106	0.21923	0.24420	0.24126	0.21962
31	Equivalent rate of discharging salt solution at mean temperature of head and tail-water, temperature of Line 9	Line 30 × Line 21 ÷ Line 24	q	cu. ft.	0.27480	0.24333	0.24735	0.24281	0.24234	0.23741	0.24114	0.24170	0.24135	0.24449	0.24120	0.21940	0.24444	0.24125	0.21962
32	Volume of salt solution discharged by pipette for stock solutions, temperature of Line 3			c.c.	9.965	9.966	9.964	9.962	9.968	9.965	9.966	9.968	9.966	9.964	9.964	9.971	9.964	9.964	9.964
33	Volume of salt solution for stock solution at mean temperature of head and tail water, temperature of Line 9	Line 32 × Line 23 ÷ Line 24		c.c.	9.971	9.965	9.965	9.974	9.972	9.968	9.957	9.962	9.973	9.969	9.964	9.972	9.968	9.964	9.972
34	Flasks used for special dilutions				1 407, 1 457	3, 22, 28, 1 407, 1 457	22, 28, 1 407, 1 457	22, 28, 1 407, 1 457	28, 1 407, 1 457	3, 28, 1 407, 1 457	3, 28, 1 407, 1 457	3, 22, 1 407, 1 457	28, 1 407, 1 457	22, 1 407, 1 457	22, 1 407, 1 457	22, 28, 1 407, 1 457	3, 22, 1 407, 1 457	3, 28, 1 407, 1 457	28, 1 407, 1 457
35	Letter denoting method of preparing special dilutions, and value of m = number of 10 c.c. pipettes of salt solution				A (m = 5)	A (m = 5)	A (m = 5)	A (m = 5)	A (m = 6)	A (m = 5)	C (m = 3)	C (m = 3)	A (m = 5)	C (m = 3)	B (m = 4)	D (m = 5)	C (m = 3)	B (m = 4)	D (m = 5)
36	Volume of normal head water for stock solutions at temperature of Line 5			c.c.	3 003.91	3 002.64	3 003.15	3 003.0	3 003.1	3 002.8	2 002.04	2 002.0	3 002.70	2 001.9	2 502.38	3 503.18	2 001.9	2 502.5	3 503.2
37	Density of normal head water for special dilutions at temperature of Line 5				0.9999	0.9998	0.9998	0.9999	0.9998	0.9999	0.9998	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9995	0.9998
38	Density of normal head water at mean temperature of head and tail water, temperature of Line 9				0.9999	0.9998	0.9998	0.9998	0.9998	0.9998	0.9999	0.9999	0.9999	0.9999	0.9999	0.9996	0.9999	0.9999	0.9999
39	Volume of normal head-water for special dilutions at mean temperature of head and tail-water, temperature of Line 9	Line 36 × (Line 37 ÷ Line 38)		c.c.	3 002.9	3 003.2	3 003.2	3 002.8	3 003.1	3 002.8	2 001.6	2 002.5	3 002.8	2 001.9	2 502.3	3 503.2	2 001.9	2 501.5	3 502.5
40	Ratio of mixture, normal head-water to salt solution for stock bottle, mean temperature of head and tail water, temperature of Line 9	Line 39 ÷ m × Line 33	r_a		60.233	60.260	60.281	60.223	60.231	60.349	67.015	66.966	66.219	66.938	62.784	....	66.944	62.763	....
41	Value of k_a		k_a		0.979	0.971	0.970	0.970	0.975	0.972	0.969	0.969	0.966	0.972	0.971	....	0.971	0.9703	....
42	Ratio of dilution of normal head-water and salt solution for stock bottles, mean temperature of head and tail-water, temperature of Line 9	Line 40 ÷ Line 41.	R_a		61.205	61.240	61.251	61.193	61.206	61.221	67.954	67.957	61.169	67.910	63.765	70.881	67.915	63.784	70.251
43	Volume discharged by pipette measuring stock solution, temperature of Line 5			c.c.	9.993	9.993	9.995	9.964	9.995	9.995	9.995	9.994	9.990	9.992	9.993	9.993	9.992	9.990	9.994
44	Density of stock solution, temperature of Line 5				1.0035	1.0035	1.0035	1.0036	1.0033	1.0034	1.0033	1.0032	1.0036	1.0031	1.0032	1.0027	1.0032	1.0031	1.0036
45	Density of stock solution, mean temperature of head and tail-water, temperature of Line 9				1.0034	1.0035	1.0036	1.0036	1.0033	1.0034	1.0033	1.0033	1.0036	1.0031	1.0035	1.0028	1.0032	1.0027	1.0028
46	Volume of stock solution when pipetted at mean temperature of head and tail-water, temperature of Line 9	Line 43 × Line 44 ÷ Line 45.		c.c.	9.994	9.993	9.994	9.994	9.994	9.993	9.994	9.994	9.990	9.992	9.991	9.992	9.992	9.993	9.993
47	Volume of flask measuring normal head-water for dilution of stock solutions, temperature of Line 5			c.c.	1 000.98	1 000.98	1 001.04	1 000.99	1 001.00	1 000.98	1 001.02	1 001.0	1 000.92	1 000.90	1 000.98	1 000.97	1 000.96	1 001.12	1 001.0
48	Volume of normal head-water for dilution of stock solutions at mean temperature of head and tail-water, temperature of Line 9	Line 47 × Line 37 ÷ Line 38.		c.c.	1 001.0	1 001.1	1 001.04	1 001.1	1 001.0	1 001.0	1 000.9	1 001.0	1 000.9	1 001.0	1 001.0	1 001.0	1 001.0	1 001.7	1 000.9
49	Ratio of dilution of normal head-water with stock solution at mean temperature of head and tail-water, temperature of Line 9	Line 48 ÷ Line 46.	R_b		100.16	100.16	100.15	100.17	100.16	100.15	100.15	100.17	100.19	100.16	100.19	100.18	100.18	100.14	100.17
50	Ratio of dilution for special dilutions at mean temperature of head and tail water, temperature of Line 9	Line 49 × Line 42.	R'		6 130.29	6 135.07	6 134.96	6 129.70	6 130.80	6 135.12	6 806.66	6 807.96	6 160.48	6 808.22	6 387.61	7 038.75	*6 803.73	6 383.38	7 037.24
51	Value of k'		k'		0.971	0.970	0.969	0.969	0.975	0.971	0.967	0.968	0.968	0.971	0.970	0.975	0.971	0.970	0.975
52	Value of $r' = R' - k'$	$R' - k'$	r'		6 129.29	6 134.06	6 133.90	6 128.73	6 129.42	6 123.15	6 607.63	6 609.28	6 189.46	6 605.29	6 386.64	7 037.77	6 602.75	6 881.35	7 036.27
53	Value of k taken same as k' except where v_2 and v'_2 differ widely		k		0.971	0.970	0.969	0.969	0.975	0.971	0.967	0.968	0.968	0.971	0.970	0.975	0.971	0.970	0.975
54	Flasks measuring special dilutions				1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457
55	Average volume of special dilutions for evaporation, temperature of Line 5			c.c.	500.48	500.49	500.03	500.49	500.51	500.49	500.51	500.68	500.46	500.48	500.49	500.49	500.48	500.30	500.50
56	Titration per sample of special dilutions evaporated			c.c.	62.903	64.820	65.211	66.478	66.958	62.101	60.620	59.995	64.454	57.294	61.050	51.716	56.367	61.415	52.153
57	Titration per liter of special dilutions evaporated at temperature of Line 5	Line 56 ÷ Line 55.		c.c.	125.645	129.513	130.287	130.824	117.796	124.083	121.136	119.870	128.798	114.478	121.980	103.331	116.630	122.693	104.202
58	Density of special dilutions at temperature of Line 5				0.9999	0.9998	0.9998	0.9998	0.9998	0.9999	0.9998	0.9998	0.9998	0.9999	0.9999	0.9999	0.9999	0.9996	0.9996
59	Density of special dilutions at mean temperature of head and tail-water, temperature of Line 9				0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999
60	Titration per liter of special dilutions at mean temperature of head and tail water, temperature of Line 9	Line 57 × Line 59 ÷ Line 58.	v'_2	c.c.	125.979	129.500	130.287	130.824	117.796	124.083	121.140	119.862	128.798	114.478	121.980	103.331	116.630	122.730	[illegible]
61	Flasks measuring dosed tail-water samples				3, 22, 23	3, 28	3	3	23	3	23	3, 22	3, 29	3, 23	3, 22	22, 28	23	23	22
62	Average volume of flasks measuring dosed tail water samples at temperature of Line 6			c.c.	500.25	500.20	500.20	500.28	500.35	500.27	500.16	500.20	500.26	500.23	500.69	500.29	500.41	500.40	500.40
63	Titration per sample of dosed tail-water in laboratory, at temperature of Line 6			c.c.	66.067	54.946	66.290	64.672	57.457	68.512	60.665	61.018	62.727	57.244	59.656	51.257	57.568	60.177	51.654
64	Titration per liter of dosed tail water at temperature of Line 6	Line 63 ÷ Line 62.		c.c.	135.946	129.840	130.572	129.279	114.834	116.361	121.291	121.959	125.889	114.404	119.643	102.469	115.012	120.258	103.225
65	Density of dosed tail-water at temperature of Line 6				0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9998	0.9999	0.9999	0.9999	0.9998	0.9999	0.9999	0.9995	0.9996
66	Density of dosed tail-water at mean temperature of head and tail-water, temperature of Line 9				0.9998	0.9998	0.9998	0.9998	0.9998	0.9999	0.9998	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999	0.9999
67	Titration per liter of dosed tail-water at mean temperature of head and tail-water, temperature of Line 9	Line 64 × Line 66 ÷ Line 65.	v_2	c.c.	136.040	129.840	130.572	129.279	114.834	116.372	121.303	121.971	125.889	114.424	119.655	102.469	115.012	120.206	103.256
68	Value of $v_2 - v'_2$	Line 67 − Line 60.	$v_2 - v'_2$		+ 10.274	+ 0.340	+ 0.285	− 1.545	− 2.962	− 7.110	+ 0.166	+ 2.089	− 4.409	− 0.054	− 2.325	− 0.862	− 3.606	− 3.434	− 0.934
69	Value of $k'v_2$ (k' used in place of k''. See Equations (96) and (100))	Line 51 × Line 67.	$k'v_2$		133.004	125.645	126.594	125.271	111.955	116.58	117.500	118.035	121.377	111.108	116.065	99.907	111.677	116.697	100.675
70	Value of $v - k'v_2$	Line 26 − Line 69.	$v - k'v_2$		601 763	616 881	638 620	628 093	551 419	590 615	639 601	624 150	617 593	586 456	590 296	633 497	594 862	604 856	535 641
71	Value of $R' \times (v_2 - v'_2)$	Line 50 × Line 68.			+ 69 283.6	+ 2 085.91	+ 1 748.45	− 9 470.30	− 16 166.3	− 43 006.5	+ 1 055.33	+ 14 220.3	− 20 896.5	− 367.374	− 14 851.2	− 6 067.40	− 24 547.8	− 18 470.3	− 6 727.60
72	Value of $R' \times (v_2 - v'_2) \div (v - k\,v_2) = f$	Line 71 ÷ Line 70.	f		+ 0.10466	+ 0.00338	+ 0.00278	− 0.01506	− 0.03293	− 0.07383	+ 0.00165	+ 0.02278	− 0.03381	− 0.00063	− 0.02478	− 0.07137	− 0.04127	− 0.03558	− 0.01256
73	Value of $1 + f$	Line 72 + 1.	$1 + f$		1.1047	1.0034	1.0028	+ 0.98494	+ 0.96707	+ 0.92617	1.00165	+ 1.02278	+ 0.96616	+ 0.99937	+ 0.97522	+ 0.92863	0.95873	0.97442	0.98744
74	Value of $r = r' \div (1 + f)$	Line 52 ÷ Line 73.	r		5 548.40	6 112.98	6 116.60	6 222.44	6 336.12	6 620.99	6 706.42	6 654.39	6 344.16	6 806.54	6 548.92	7 113.71	7 095.50	6 548.67	7 125.77
75	Value of $Q = q\,r$	Line 31 × Line 74.	Q		1 524.70	1 487.69	1 512.96	1 510.87	1 535.99	1 571.69	1 638.30	1 608.44	1 531.16	1 664.12	1 579.80	1 561.81	1 734.41	1 579.97	1 563.80
76	Value of k	Same as Line 53.	k		0.971	0.970	0.969	0.969	0.975	0.971	0.967	0.968	0.969	0.971	0.970	0.975	0.971	0.970	0.975
77	Value of $k\,q$	Line 31 × Line 76.	$k\,q$		0.267	0.236	0.240	0.235	0.236	0.231	0.233	0.234	0.234	0.237	0.234	0.214	0.237	0.234	0.214
78	Value of $Q_2 = Q + k\,q$	Line 75 + Line 77.	Q_2		1 525.0	1 487.6	1 513.2	1 511.1	1 536.2	1 572.1	1 639.1	1 608.7 Mean	1 531.4 A and B Tests	1 664.4 / 1 670.4	1 579.8 / 1 580.0	1 562.1 / 1 562.6	1 675.2	1 580.1	1 563.6

NOTE.—* Test 120 B:—R' (line 50) seems to be in error. Computations for R'. Formula, $R' = \frac{V - kv_1}{v_2' - V_1}$. Value of v_1 is not given and was computed from Test 120 A as follows: $v_1 = \frac{R' v_2' - v}{r'}$ or $v_1 = \frac{6\,808.2 \times 114.478 - 586\,567}{6\,802} = 28.26$. Then $R' = \frac{594\,964 - 0.97 \times 28.26}{118.620 - 28.26} = 6\,584.07$ (Test 120 B). From which Q_2 = 1 676.9. Dosed Tail Sampling Pipes on Upper Section.

the salt-solution and special dilution titrations, no correction is necessary, except where there is a large relative difference, say 15% or even 20%, between the titrations of the salt-solution and special dilution samples.

Test 105, September 12th, 1914.

Titration, September 13th.

Nitrate (1.56 grammes), September 8th.

Dilute salt solution. Temp. 18° cent. Flasks Nos. 1 456-1 474.		Normal head-water. Temp. 18.25° cent. Flasks Nos. 3-11.	Special dilutions.* Temp. 18.25° cent. Flasks Nos. 1 456-1 474.		
5 310	5 350	1 310	4 775	4 820	9 595
5 350	5 355	1 305	4 835	4 805	9 640
5 350	5 360	1 320	4 810	4 790	9 600
5 355	5 360	1 315			
5 363	5 352	1 305			
5 365	5 347	1 320			
Mean = 53.514		Mean = 13.125	Mean = 48.058		

Tail-Water Titrations.
Temp. 18.25° cent. Flask No. 11.

5 055		5 045		5 210	
	5 050		5 160		5 260
5 070		5 150		5 235	
	6 208		6 700		6 770
6 230		6 713		6 770	
	6 470		6 910		6 918

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	50 740	52 162	51.372
Lower	64 052	68 136	66.321
Averages	57.396	60.149	58.847 = Arith. mean.

Weighted mean = 57.920

* Four 10-c.c. pipettes of salt solution from Samples ———, 40 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c with normal head-water This liter then divided into two nearly equal parts, 500 c.c. each. for evaporation to about 10 c.c. in a casserole for titration.

Test 106, September 15th, 1914.

Titration. September 16th.

Nitrate (1.56 grammes), September 8th.

Dilute salt solution. Temp 22° cent Flasks Nos. 1 456-1 474.		Normal head-water. Temp. 18° cent. Flasks Nos. 3-11.	Special dilutions.* Temp. 21° cent. Flasks Nos. 1 456-1 474.		
4 340	4 335	1 305	4 150	4 150	8 300
4 335	4 340	1 309	4 120	4 165	8 285
4 335	4 335	1 305	4 190	4 095	8 285
4 335	4 335	1 302	4 147	4 135	8 282
4 335	4 335	1 295			
4 330	4 330	1 280			
Mean = 43.350		Mean = 12.993	Mean = 41.440		

Tail-Water Titrations.
Temp. 21° cent. Flask No. 3.

4 650		4 725		4 615	
	4 730		4 710		4 720
4 690		4 750		4 775	
	4 855		4 880		4 945
4 435		4 710		4 925	
	4 560		4 905		4 950

Calculations of Averages for Quarters and Halves of Race.

	West.	East	Averages.
Upper.....	4 709	4 705	47.072
Lower.....	4 640	4 921	47.961
Averages.	46.745	48.130	47.517 = Arith. mean.

Weighted mean = 47.087

* Four 10-c.c. pipettes of salt solution from Samples ———, 40 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c each, for evaporation to about 10 c.c. in a casserole for titration.

Test 107, September 16th, 1914.

Titration, September 17th.

Nitrate (1.56 grammes), September 8th.

Dilute salt solution.* Temp. 19.5° cent. Flasks Nos. 1 456–1 474.		Normal head-water. Temp. 21° cent. Flasks Nos. 3-11.	Special dilutions.* Temp. 21° cent. Flasks Nos. 1 456–1 474.		
2 980	2 975	1 295	3 240	3 240	6 480
2 975	2 975	1 290	3 270	3 220	6 490
2 985	2 985	(1 350)	3 242	3 235	6 477
2 975	2 978	1 320	3 240	3 225	6 465
		(1 377)			
		1 310			
Mean = † 29.755		Mean = 13.038	Mean = 32.390		

Tail-Water Titrations.

Temp. 20.5° cent. Flask No. 3.

4 055		4 015		3 975	
	4 085		4 070		4 010
4 090		4 075		4 025	
	4 511		4 757		4 795
4 345		4 660		4 800	
	4 508		4 837		4 855

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	40 640	40 200	40.444
Lower.....	45 060	48 088	46.742
Averages.	42.850	44.144	43.593 = Arith. mean.

Weighted mean = 43.209

* Four 10-c.c. pipettes of salt solution from Samples———, 40 c.c., diluted with 3 000 c.c. normal head water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† 0.03 c.c. of $AgNO_3$ deducted from salt solution titration because that much was used up by the $K_2Cr_2O_7$, as the concentration of salt solution was weak for the ratio of dilution that was used in the laboratory on January 3d, 1915.

Test 108, September 17th, 1914.

Titration, September 19th.

Nitrate (1.56 grammes), September 8th.

Dilute salt solution. Temp. 23° cent. Flasks Nos. 1 456–1 474.		Normal head-water. Temp. 19° cent. Flask No. 11.		Special dilutions.* Temp. 22.5° cent. Flasks Nos. 1 456–1 474.		
5 630	5 630	1 305	1 295	4 160	4 047	8 207
5 637	5 635	1 300	1 290	4 143	4 075	8 218
5 630	5 630	1 295	1 290	4 100	4 085	8 185
5 625	5 625			4 065	4 050	8 115
Mean = 56.303 †		Mean = 12.958		Mean = 40.906		

Tail-Water Titrations.

Temp. 19° cent. Flask No. 11.

4 951		4 910		4 525	
	4 905		4 813		4 516
4 875		4 854		4 730	
	5 130		4 920		4 880
4 835		4 900		4 900	
	4 850		4 905		4 925

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	48 990	46 460	47.865
Lower.....	49 288	49 060	49.161
Averages.	49.139	47.760	48.513 = Arith. mean.

Weighted mean = 48.778

* 40 c.c. salt solution from Sample *I*, diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† Four 10-c.c. pipettes of salt solution diluted to 3 000 c.c. distilled water.

Test 109, September 18th, 1914.

Titration, September 19th.

Nitrate (1.56 grammes), September 8th.

Dilute salt solution. Temp. 20° cent. Flasks Nos. 1 456-1 474.		Normal head-water, Temp. 20° cent. Flask No. 11.		Special dilutions.* Temp. 20° cent. Flasks Nos. 1 456–1 474.		
6 115	6 125	1 290	1 320	4 375	4 305	8 680
6 127	6 130	1 300	1 310	4 375	4 362	8 737
6 130	6 140	1 320	1 320	4 300	4 360	8 660
6 130	6 130			4 315	4 390	8 705
Mean = 61.284†		Mean = 13.100		Mean = 43.478		

*40 c.c. salt solution from Sample *I*, diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

†Four 10-c.c. pipettes of salt solution diluted to 3 000 c.c. distilled water.

Tail-Water Titrations.

Temp. 22.5° cent. Flask No. 11.

5 170		5 130		4 790	
	5 150		5 170		4 625
5 130		5 130		4 870	
	5 140		5 057		5 048
5 045		5 058		5 042	
	5 045		5 060		5 070

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper	51.425	48 638	50.183
Lower	50 720	50 554	50.628
Averages.	51.072	49.596	50.406 = Arith. mean.

Weighted mean = 50.641

The pipette and flask calibrations are given in Figs. 4 and 6, respectively.

108.—Tests on Which Special Dilutions Were Used.—No special dilutions were used in the tests on Unit No. 13, except in the second treatments of Tests 23 and 26. In all the tests on Unit No. 12, and in the first series on Unit No. 11, special dilutions and normal head-water samples were both used so as to show that the corrections computed in Section 43 give accurate results when applied to normal head-water titrations. A comparison of the simultaneous results by the two methods is given in Section 86.

No normal head-water samples were titrated in the second series of tests on Unit No. 11.

109.—Final Computations.—The final computations, individually unadjusted except as to the calibrations of instruments and corrections indicated in the various tables, appear in Tables 54, 55, 56, 57, 58, and 59.

Table 54, Unit No. 13, First Series.—This is for the first, or lettered, series of tests on Unit No. 13. These tests are quite accurate as to discharge measurements, but cannot be regarded as anything but

preliminary tests for the purpose of adjusting the apparatus and training the corps of observers. The power was measured on the switchboard instruments only.

Test 111, November 9th, 1914.

Titration, November 10th.

Nitrate (1.45 grammes), November 6th.

Dilute salt solution. Temp. 6.2° cent. Flasks Nos. 1 407–1 457.		Special dilutions.* Temp. 5.8° cent. Flasks Nos. 1 407–1 457.		
6 000	6 000	6 266	6 231	12 497
5 990		6 340	6 362	12 702
6 005	6 005	6 254	6 289	12 543
5 988	5 980			
6 000	5 982			
Mean = 59.944		Mean = 62.903		

Tail-Water Titrations.

Temp. 8.3° cent. Flasks Nos. 3–22–28.

6 490		6 445		6 155	
	6 424		6 157		6 222
6 219		6 115		6 222	
	7 443		7 208		7 200
7 284		7 288		7 258	
	7 220		7 316		7 287

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	63 386	61 890	62.721
Lower.....	73 088	72 538	72.782
Averages.	68.237	67.214	67.752 = Arith. mean.
Weighted mean = 68.007			

* Five 10-c.c. pipettes of salt solution from Samples *A* and *E*. 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

Test 112, November 10th, 1914.

Titration, November 11th.

Nitrate (1.45 grammes), November 6th.

Dilute salt solution. Temp. 6.9° cent. Flasks Nos. 1 407–1 457.		Special dilutions.* Temp. 8° cent. Flasks Nos. 1 407–1 457.		
6 141	6 152	6 480	6 437	12 917
6 186	6 187	6 498	6 478	12 976
6 150	6 163	6 455	6 564	13 019
6 160	6 171	6 464	6 480	12 944
6 128	6 137			
Mean = 61.575		Mean = 64.820		

Tail-Water Titrations.

Temp. 8° cent. Flasks Nos. 3–28.

5 914		5 830		6 087	
	5 866		5 990		7 100
5 862		5 786		6 115	
	6 827		7 068		7 353
7 135		7 382		7 420	
	6 328		7 450		7 510

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper	58 516	63 230	60.611
Lower	69 180	73 602	71.637
Averages.	63.848	68.416	66.124 = Arith. mean.
Weighted mean = 64.946			

* Five 10-c.c. pipettes of salt solution from Samples ———, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

Table 55, Unit No. 13, Second Series.—In these tests the power was measured by the high-precision, direct-current instruments very satisfactorily, but there was some question toward the last as to the tail-

Test 113, November 11th, 1914.

Titration, November 12th.

Nitrate (1.45 grammes), November 6th.

Dilute salt solution. Temp. 4° cent. Flasks Nos. 1 407–1 457.		Special dilutions.* Temp. 8.5° cent. Flasks Nos. 1 407–1 457.		
6 270	6 280	6 522	6 512	13 034
6 264	6 252	6 492	6 512	13 004
6 289	6 290	6 540	6 530	13 070
6 264	6 270	6 539	6 522	13 061
6 260	6 265			
Mean = 62.695		Mean = 65.211		

Tail-Water Titrations.
Temp. 9° cent. Flask No. 3.

5 897		5 915		6 133	
	5 868		5 912		6 120
5 828		5 828		5 983	
	7 206		7 200		7 352
7 036		7 205		7 444	
	6 917		7 428		7 423

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	58 672	60 370	59.427
Lower.....	70 910	73 694	72.457
Averages.	64.791	67.032	65.942 = Arith. mean.

Weighted mean = 65.320

* Five 10-c.c pipettes of salt solution from Samples —— and *E*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c of this diluted to 1 000 c c. with normal head-water. This liter then divided into two nearly equal parts, 500 c c. each, for evaporation to about 10 c.c. in a casserole for titration.

Test 114, November 12th, 1914.

Titration, November 13th.

Nitrate (1.45 grammes), November 12th.

Dilute salt solution. Temp. 6° cent. Flasks Nos. 1 407–1 457.		Special dilutions.* Temp. 4° cent. Flasks Nos. 1 407–1 457.		
6 267	6 277	6 550	6 535	13 085
6 272	6 271	6 538	6 546	13 084
6 260		6 581	6 527	13 108
6 260	6 270	6 576	6 528	13 104
6 255	6 265			
Mean = 62.663		Mean = 65.476		

Tail-Water Titrations.
Temp. 8.5° cent. Flask No. 3.

5 960		5 957		6 027	
	5 912		5 910		6 020
5 908		5 814		5 990	
	6 928		7 020		7 304
6 795		7 145		7 410	
	6 745		7 400		7 396

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	59 102	59 868	59.442
Lower.....	69 033	73 060	71.270
Averages.	64.068	66.464	65.356 = Arith. mean.

Weighted mean = 64.672

* Five 10-c.c. pipettes of salt solution from Samples *A* and *C*, 50 c.c., diluted with 3 000 c c. normal head water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

water samples, owing to the discovery of leaks in the tail-water sampling system, in the lower section, at the elevation of the river water above the tail-water proper. This led to the transfer of the twelve tail-water sampling pipes from the lower to the upper sampling section, and to an increase in the number of regular samples from twelve to eighteen.

Test 115, November 13th, 1914.

Titration, November 14th.

Nitrate (1.45 grammes), November 12th.

Dilute salt solution. Temp. 7.6° cent. Flasks Nos. 1 407-1 457.		Special dilutions.* Temp. 6.4° cent. Flasks Nos. 1 407-1 457.		
5 485	5 494	5 908	5 880	11 788
5 514	5 500	5 860	5 881	11 741
5 475	5 510	5 935	5 905	11 840
5 495	5 477	5 921	5 876	11 797
5 480	5 514			
Mean = 54.944		Mean = 53.958		

Tail-Water Titrations.
Temp. 8.5° cent. Flask No. 22.

5 424		5 400		5 429	
	5 365		5 320		5 488
5 312		5 260		5 404	
	5 956		6 137		6 462
5 949		6 185		6 500	
	5 950		6 517		6 560

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper....	53 522	54 102	53.780
Lower.....	60 160	64 352	62.462
Averages.	56.811	59.227	58.121 = Arith. mean.

Weighted mean = 57.457

*Five 10-c.c. pipettes of salt solution from Samples A and D, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c., with normal head-water. This liter then divided into two nearly equal parts, 500 c. c. each, for evaporation to about 10 c.c. in a casserole for titration.

Test 116, November 14th, 1914.

Titration. November 16th.

Nitrate (1.45 grammes), November 12th.

Dilute salt solution. Temp. 13.5° cent. Flasks Nos. 1 407-1 457.		Special dilutions.* Temp. 6° cent. Flasks Nos. 1 407-1 457.		
5 860	5 877	6 215	6 220	12 435
5 875	5 872	6 186	6 213	12 399
5 875	5 864	6 248	6 173	12 421
5 875	5 878	6 178	6 248	12 426
5 855	5 883			
Mean = 58.714		Mean = 62.101		

Tail Water Titrations.
Temp. 9.5° cent. Flask No. 3.

5 650†		5 684		5 534	
	5 617		5 558		5 578
5 577		‡5 519		‡5 519	
	6 009		6 151		6 437
5 800		6 187		6 464	
	5 998		6 348		6 500

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	56 094	55 472	55.818
Lower.....	59 985	63 800	62.104
Averages.	58.039	59.636	58.961 = Arith. mean.

Weighted mean = 58.512

*Five 10-c.c. pipettes of salt solution from Samples A and D, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† Interpolated value—spoiled titration.

‡ Titrated together.

Table 56, Unit No. 13, Third Series.—This is the best series of tests on any of the three units. The power was accurately measured by the high-precision, direct-current instruments.

The tail-race samples were very satisfactory, having been taken from eighteen points in the upper sampling station.

Test 117, November 16th, 1914.

Titration, November 17th.

Nitrate (1.45 grammes), November 12th.

Dilute salt solution. Temp. 9° cent. Flasks Nos. 1 407-1 476.		Special dilutions.* Temp. 9° cent. Flasks Nos. 1 407-1 457.		
6 365	6 353	6 090	6 043	12 133
6 367	6 370	6 072	6 060	12 132
6 377	6 392	6 010	6 075	12 085
6 358	6 353	6 032	6 122	12 154
6 345	6 345			
Mean = 63.625		Mean = 60.630		

Tail-Water Titrations. Temp. 8.2° cent. Flask No. 28.

5 900	5 783	5 828	5 717	5 680	5 800
5 783		5 670		5 800	
	5 885		6 486		6 670
6 127		6 520		6 720	
	6 320		6 668		6 665

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	57 928	57 492	57.734
Lower.....	62 130	66 418	64.512
Averages.	60.029	61.955	61.123 = Arith. mean.
Weighted mean = 60.665			

* Three 10-c.c. pipettes of salt solution from Samples *A* and *D*, 30 c.c., diluted with 2 000 c.c. normal head-water, and 10 c.c. of this diluted with 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

Test 118, November 17th, 1914.

Titration, November 18th.

Nitrate (1.45 grammes), November 12th.

Dilute salt solution. Temp. 7° cent. Flasks Nos. 1 407-1 457.		Special dilutions.* Temp. 7° cent. Flasks Nos. 1 407-1 457.		
6 239	6 215	6 005	5 991	11 996
6 240	6 230	6 025	6 015	12 040
6 205	6 190	5 965	6 025	11 990
6 231	6 215	5 970	6 000	11 970
6 206	6 170			
Mean = 62.141		Mean = 59.995		

Tail-Water Titrations. Temp. 8° cent. Flasks Nos. 3-22.

6 375		6 335		6 140	
	6 169		6 172		6 298
6 250		6 123		6 252	
	5 765		5 734		6 375
5 819		5 959		6 255	
	5 965		6 140		6 277

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	62 504	62 155	62.349
Lower.....	58 770	61 562	60.321
Averages.	60.637	61.858	61.335 = Arith. mean.
Weighted mean = 61.016			

* Three 10-c.c. pipettes of salt solution from Samples *A* and *D*, 30 c.c., diluted with 2 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each for evaporation to about 10 c.c. in a casserole for titration.

It should be remarked that Test 39 has been interpolated. The head-water elevations were lost, and, unfortunately, there was an unusual variation in this respect during the test. This was a disappointment, as it would have been desirable to know what degree of consistency such a test would show, as compared with those made under more favorable conditions. As it is, very little weight should be at-

Test 119, November 18th, 1914.

Titration. November 19th.

Nitrate (1.45 grammes), November 12th.

Dilute salt solution. Temp. 2° cent. Flasks Nos. 1 407–1 457.		Special dilutions.* Temp. 4.2° cent. Flasks Nos. 1 407–1 457.		
6 130	6 147	6 420	6 468	12 888
6 185	6 180	6 390	6 455	12 845
6 131	6 145	6 478	6 430	12 908
6 168	6 150	6 435	6 490	12 925
6 157	6 145			
Mean = 61.538		Mean = 64.458		

Tail-Water Titrations.
Temp. 4.6° cent. Flasks Nos. 3–32.

6 649		6 650		6 195	
	6 598		6 385		6 165
6 475		6 395		6 170	
	6 295		5 875		5 930
6 253		5 867		5 821	
	6 126		5 845		5 805

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper	65 534	62 288	64.091
Lower ...	61 352	58 552	59.797
Averages	63.443	60.420	61.944 = Arith. mean.

Weighted mean = 62.727

* Five 10-c.c. pipettes of salt solution from Samples A and E, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

Test 120-A, November 19th, 1914.

Titration. November 20th.

Nitrate (1.45 grammes), November 19th.

Dilute salt solution. Temp. 4.2° cent. Flasks Nos. 1 407–1 457.		Special dilutions.* Temp. 4° cent. Flasks Nos. 1 407–1 457.		
5 845	5 845	5 738	5 732	11 470
5 840	5 840	5 725	5 735	11 460
5 840	5 853	5 750	5 710	11 460
5 850	5 833	5 744	5 701	11 445
Mean = 58.429		Mean = 57.294		

Tail-Water Titrations.
Temp. 6.1° cent. Flasks Nos. 3–22.

5 968		5 980		5 849	
	5 895		5 780		6 004
5 834		5 705		5 585	
	5 678		5 503		5 765
5 547		5 425		5 684	
	5 660		5 510		5 685

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper	58 764	58 045	58.444
Lower	55 775	56 294	56.063
Averages	57.269	57.170	57.254 = Arith. mean.

Weighted mean = 57.244

* Three 10-c. c. pipettes of salt solution from Samples A and D, 30 c.c., diluted with 2 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c. c. each, for evaporation to about 10 c.c. in a casserole for titration.

tached to this test. The method of interpolating was to determine a head-water elevation which would satisfy the facts and agree with the other tests in a satisfactory manner.

Table 57, Unit No. 12.—These tests are not as satisfactory as the Third Series on Unit No. 13, there being some irregularities in both the power measurements and tail-water samples. The irregularities in

Test 120-*B*, November 19th, 1914.

Titration, November 22d.

Nitrate (1.45 grammes), November 19th.

Dilute salt solution. Temp. 20° cent. Flasks Nos. 1 407–1 457.		Special dilutions.* Temp. 4° cent. Flasks Nos. 1 407–1 457.		
5 914	5 920	5 929	5 960	11 889
5 918	5 913	5 931	5 940	11 871
5 899	5 903	5 960	5 900	11 860
5 900	....			
5 875	5 847			
Mean = 58.988		Mean = 59.367		

Tail-Water Titrations.

Temp. 13.6° cent. Flask No. 22.

6 000		5 977		5 897	
	5 905		5 825		6 045
5 860		5 758		....	
	5 696		5 555		5 731
5 603		5 440		5 640	
	5 660		5 518		5 603

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	59 000	59 223	59.084
Lower.....	55 998	56 094	56.051
Averages.	57.499	57.658	57.478 = Arith. mean.

Weighted mean = 57.553.

* Three 10-c.c. pipettes of salt dilution from Samples *A* and *D*, 30 c.c., diluted with 2 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

Test 121-*A*, November 20th, 1914.

Titration, November 21st.

Nitrate (1.45 grammes), November 19th.

Dilute salt solution. Temp. 5° cent. Flasks Nos. 1 407–1 457.		Special dilutions.* Temp. 4.6° cent. Flasks Nos. 1 407–1 457.		
5 966	5 975	6 100	6 140	12 240
5 972	5 978	....	(6 092)	....
5 965	5 948	6 120	6 080	12 200
5 980	5 976	6 115	6 075	12 190
5 965	5 960			
Mean = 59.685		Mean = 61.050		

Tail-Water Titrations.

Temp. 7.2° cent. Flasks Nos. 3–22.

6 645		6 618		6 285	
	6 450		6 278		6 394
6 470		6 260		6 450	
	5 955		5 271		5 344
5 582		5 365		5 253	
	5 550		5 320		5 215

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	64 886	63 518	64.278
Lower.....	56 130	52 806	54.283
Averages.	60.508	58.162	59.280 = Arith. mean.

Weighted mean = 59.856.

* Four 10-c.c. pipettes of salt solution from Samples *A* and *D*, 40 c.c., diluted with 2 500 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each. for evaporation to about 10 c.c. in a casserole for titration.

the power measurements may be attributed largely to a change in the personnel of the power observers and manipulators, and to the greater haste necessary in order to complete the work within the time limit of 2 weeks for this unit. The discrepancies in the discharge measurements result largely from uncertainties as to the proper weights for the various tail-water concentrations, which become relatively more important for Unit No. 12 owing to the fact that the hydraulic condi-

Test 121-*B*, November 20th, 1914.

Titration, November 26th.

Nitrate (1.45 grammes), November 23d.

Dilute salt solution. Temp. 20° cent. Flasks Nos. 1 407–1 457.		Special dilutions.* Temp. 4.5° cent. Flasks Nos. 1 407–1 457.		
6 030	6 013	6 093	6 147	12 240
5 972	5 952	6 122	6 138	12 260
6 026	6 015	6 165	6 191	12 356
5 958	5 956	6 065	6 211	12 276
6 022	6 019			
Mean = 59.963		Mean = 61.415		

Tail-Water Titrations.
Temp. 12.9° cent. Flask No. 22.

6 700		6 622		6 319	
	6 490		6 342		6 430
6 489		6 308		6 495	
	5 970		5 311		5 373
5 605		5 371		5 295	
	5 600		5 360		5 265

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	65 218	63 965	64.661
Lower......	56 365	53 208	54.611
Averages.	60.792	58.586	59.636 = Arith. mean.

Weighted mean = 60.177

*Four 10-c.c. pipettes of salt solution from Samples *A* and *D*, 40 c.c., diluted with 2 500 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

Test 122, November 21st, 1914.

Titration, November 22d.

Nitrate (1.45 grammes), November 19th.

Dilute salt solution. Temp. 2° cent. Flasks Nos. 1 407–1 457.		Special dilutions.* Temp. 2° cent. Flasks Nos. 1 407–1 457.		
5 344	5 320	5 145	5 140	10 285
5 323	5 310	5 155	5 149	10 304
5 323	5 305	5 215	5 175	10 390
5 305	5 305	5 194	5 200	10 394
5 318	5 310			
Mean = 53.163		Mean = 51.716		

Tail-Water Titrations.
Temp. 6° cent. Flasks Nos. 22–28.

5 190		(8 953)		5 060	
	(8 377)		(8 130)		5 159
4 940		5 340		5 080	
	(8 082)		(7 070)		(7 902)
5 130		4 988		(6 801)	
	5 195		(6 540)		(6 527)

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	51 566	50 997	51.282
Lower.....	51 043	†51 202	51.122
Averages.	51.304	51.100	51.202 = Arith. mean.

Weighted mean = 51.257

* *Special Dilution D.*—Five 10-c.c. pipettes of salt solution from Samples *A*, *B*, *C*, *D*, and *E*, 50 c.c., diluted to 3 500 c.c. with normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† Value interpolated.

TABLE 54.—FINAL COMPUTATIONS. TESTS OF UNIT NO. 13. TESTS *A*-*W*. FIRST SERIES. JULY 22D–AUGUST 13TH, 1914.

Missing Letters: *M* Represents a Distribution Test. *A*, *D*, and *E* Represent Preliminary Salt Tests. *B*, *C*, *F*, *J*, *P*, *Q*, *R* Represent Current-Meter Tests.

Date.	Test.	Duration, in minutes.	Gate Opening, in Inches.		Speed, in revolutions per minute.	Head, h, in feet.	Power at machine terminals, in kilowatts.	Discharge, in cubic feet per second.	Performance on 30 Ft. Head.				Over-all efficiency on Unit No. 13, in percentage.	Mean temperature of water, in degrees, centigrade.	Weight of water, in pounds per cubic foot.	Remarks.
			On reg. cyl.	Actual.					Speed, in revolutions per minute.	General efficiency = 100%. Power at mach. ter., in kilowatts.	General efficiency = 100%. Power at mach. ter., in horse-power.	Discharge, in cubic feet per second.				
1914																
8-10	*U*	30-0	13½	5.00	105.0	33.95	3 622	1 440	98.7	3 008	4 032	1 354	87.58	18.8	62.334	Shower before test started. Rough calculations indicate that the salt solutions of Tests *U* and *T* were interchanged.
8-12	*V*	30-30	13½	5.00	102.0	34.17	3 666	1 553	95.6	3 015	4 042	1 455	81.71	18.4	62.338	
8-13	*W*	20-0	14	5.24	101.8	33.41	3 681	1 556	96.5	3 132	4 198	1 474	83.77	18.8	62.334	
8-8	*S*	30-0	14⅛	5.48	108.7	35.08	4 055	1 640	100.5	3 207	4 299	1 517	83.35	18.3	62.340	In this series of tests: All samples are titrated separately. Dosing pipes not plugged in middle. The power was measured by the switchboard instruments. There was some question as to the tail-water samples during some of the tests, owing to the discovery of leaks in the sampling pipes.
7-24 P. M.	*L*	15-50	14.55	5.50	108.6	35.05	4 083	1 647	100.5	3 233	4 334	1 524	83.64	19.4	62.326	
7-22	*G*	24-45	"	"	108.0	35.10	4 078	1 662	99.8	3 222	4 319	1 536	82.70	19.1	62.330	
7-24 A. M.	*H*	16-0	"	"	108.0	35.24	4 112	1 662	99.7	3 230	4 330	1 534	83.02	19.3	62.327	
7-24 A. M.	*I*	16-0	14.6	5.52	107.8	35.30	4 115	1 666	99.4	3 224	4 322	1 536	82.76	19.4	62.326	
7-24 P. M.	*K*	18-0	14.55	5.50	103.1	34.97	4 069	1 683	100.1	3 233	4 334	1 558	81.82	19.4	62.326	Dosed tail sampling pipes on lower section.
8-10	*T*	45-0	14⅛	5.48	108.7	35.36	4 120	1 746	100.1	3 220	4 316	1 609	78.89	18.6	62.336	Discharge obtained by method of unbalanced evaporations.
7-30	*O*	39-0	16⅛	6.26	108.0	34.98	4 333	1 768	100.0	3 442	4 614	1 637	82.90	18.6	62.336	
7-28	*N*	89-0	16⅛	6.26	107.8	34.53	4 234	1 777	100.5	3 429	4 597	1 656	81.65	19.0	62.331	This unit generates direct current.
														Ave..	62.332	Rough calculations indicate that the salt solutions of Tests *U* and *T* were interchanged.

TABLE 55.—(*Continued.*)

Date.	Test	Duration, in minutes.	Gate Opening, in Inches. On reg. cyl.	Gate Opening, in Inches. Actual.	Speed, in revolutions per minute.	Head, h, in feet.	Net power at machine terminals, in kilowatts.	Discharge, in cubic feet per second.	Performance on 30-Ft. Head. Speed, in revolutions per minute.	Performance on 30-Ft. Head. General efficiency = 100%. Power at machine terminals, in kilowatts.	Performance on 30-Ft. Head. General efficiency = 100%. Power at machine terminals, in horse power.	Performance on 30-Ft. Head. Discharge, in cubic feet per second.	Over-all efficiency on Unit No. 13, in percentage.	Mean temperature of water, in degrees, centigrade.	Weight of water, in pounds per cubic foot.	Remarks.
1914. 8-15	10	30	14	5.24	105.0	33.75	3 767	1 586 1 579	99.0	3 157	4 232	1 496 1 488	83.20 83.65	19.0	62.331	Showers at times. Apparently a good test chemically. All samples titrated separately.
8-21	20	18	14½	5.48	97.5	32.87	3 660	1 595 1 581	93.1	3 192	4 279	1 524 1 510	82.58 83.35	18.8	62.334	½ test chemically. E. and W. D. T. samples combined.
8-21	21	18	14½	5.48	101.8	32.80	3 681	1 589 1 582	97.4	3 219	4 315	1 520 1 513	83.50 83.88	18.8	62.334	½ test chemically. E. and W. D. T. samples combined, except Nos. 21 and 22, titrated separately.
8-20	18	18	15	5.72	117.1	33.61	3 914	1 636 1 609	110.6	3 301	4 425	1 546 1 520	84.18 85.62	18.6	62.336	Head-water muddy. Samples 21 and 22 titrated separately. Combined 23 with 24, 25 and 27, 26 and 28 29 and 31, and 30 and 32. No. 22 bad sample.
8-18	11	30	15	5.72	106.1	34.11	4 018	1 661 1 652	99.5	3 314	4 442	1 558 1 549	83.86 84.34	18.8	62.334	Slight rain during test. Apparently good test chem. Samples titrated separately. No. 22 doubtful.
8-20	17	18	15	5.72	111.1	33.63	3 980	1 650 1 645	104.9	3 353	4 495	1 559 1 554	84.80 85.07	18.6	62.336	Head-water muddy. Samples 21 and 22 titrated separately. Combined 23 and 24, 25 and 27, 26 and 28, 29 and 31, 30 and 32. No. 22 bad sample.
8-18	12	30	15½	5.96	107.2	34.83	4 220	1 731 1 726	99.5	3 373	4 522	1 606 1 601	82.82 83.07	18.8	62.334	Apparently good test chem. Samples titrated separately. No. 22 doubtful.
														Ave.=	62.333	used on this series of tests.

General Notes.—This unit generates direct current. The flow of salt solution was not stopped between Tests 7 and 8, 13 and 14, 15 and 16, 17 and 18, 20 and 21, and each pair of tests was run in immediate succession. "East and West Dosed Tail Samples Mixed" means that each sample taken on the east side of draft-tube was mixed with its corresponding west sample taken at same elevation. Thus there were half as many samples to titrate as were actually taken out of the draft-tube.

In these tests the power was measured by the high-precision, direct-current instruments, and was very satisfactory, but there was some question as to the tail-water samples during some of the tests, owing to the discovery of leaks in the sampling pipes in the lower section of the elevation of river water above the tail-water proper. The upper figures in discharge and efficiency columns, are results obtained by taking titration of doubtful samples as correct. The lower figures are obtained by omitting the titration of doubtful samples, values being substituted, instead, which are based on the titration of adjacent samples.

Dosed tail-sampling pipes on lower section. Eight samples on upper section also in Test 19. Gate locked on all tests.

Discharge obtained by method of unbalanced evaporations.

TABLE 55.—FINAL COMPUTATIONS. TESTS OF UNIT NO. 13. TESTS 6-21, INCLUSIVE. SECOND SERIES.
AUGUST 13TH–AUGUST 21ST, 1914.
Missing Tests, 1-5 Inclusive, Represent Current-Meter Tests.

Date.	Test.	Duration, in minutes.	Gate Opening, in Inches. On reg. cyl.	Gate Opening, in Inches. Actual.	Speed, in revolutions per minute.	Head, h, in feet.	Net power at machine terminals, in kilowatts.	Discharge, in cubic feet per second.	Performance on 30-Ft. Head. Speed, in revolutions per minute.	Performance on 30-Ft. Head. General efficiency = 100%. Power at machine terminals, in kilowatts.	Performance on 30-Ft. Head. General efficiency = 100%. Power at machine terminals, in horse-power.	Performance on 30-Ft. Head. Discharge, in cubic feet per second.	Over-all efficiency on Unit No. 13, in percentage.	Mean temperature of water, in degrees, centigrade.	Weight of water, in pounds per cubic foot.	Remarks.
1914.																
8-19	13	18	12⅞	4.70	97.2	33.84	3 417	1 424 1 431	9.15	2 852	3 823	1 340 1 347	83.91 83.48	18.8	62.334	Head-water muddy. ⅓ test chem. E. and W. D. T. samples mixed. Salt soln. not turned off between Tests 13 and 14.
8-19	14	18	12⅞	4.70	102.0	33.92	3 420	1 438	96.0	2 845	3 814	1 353	82.91	18.8	62.334	Head water muddy. ½ test chemically. E and W. D. T. samples mixed.
8-15	9	28	12⅞	4.70	107.0	34.42	3 548	1 460	99.9	2 887	3 870	1 363	83.51	19.2	62.329	Apparently a good test chem. Samples titrated separately.
8-14	8	20	13½	5.00	113.0	34.60	3 762	1 522 1 514	105.2	3 037	4 071	1 417 1 410	84.50 84.92	19.0	62.331	Rain before test. Weighting imperfect. ⅓ test chemically. E. and W. D. T. samples mixed.
8-20	19	30	13½	5.00	106.0	33.69	3 612	1 517 1 507	100.0	3 035	4 068	1 432 1 422	83.55 84.14	18.8	62.334	Head-water very dirty. Apparently good test chemically. Samples titrated separately. 12 D. T. samples on lower sec. 8 on upper.
8-14	7	20	13½	5.00	100.0	34.72	3 851	1 556	93.0	3 093	4 146	1 446	84.33	19.0	62.331	Rain before test. ⅓ test chemically. E. and W. D. T. samples mixed.
8-19	16	18	14	5.24	112.0	32.81	3 559	1 526	107.1	3 111	4 170	1 459	84.06	18.9	62.333	Head-water muddy. ⅓ test chemically. E. and W. D. T. samples mixed.
8-19	15	18	14	5.24	108.0	32.88	3 585	1 555 1 548	103.1	3 125	4 189	1 486 1 478	82.91 83.36	18.9	62.333	½ test chemically. E. and W. D. T. samples mixed.
8-13	6	20	14	5.24	103.1	33.65	3 760	1 571	97.4	3 165	4 243	1 483	84.15	19.2	62.329	Rain before test Apparently a good test chemically. All samples titrated separately.

TABLE 56.—Final Computations. Tests of Unit No. 13. Tests 22-46, Inclusive. Third Series. August 21st–September 2d, 1914.

Missing Tests, 24, 25, 27-31, Inclusive, and 34-38, Inclusive, Represent Current-Meter Tests.

Date.	Test.	Duration, in minutes.	Gate Opening, in Inches.		Speed, in revolutions per minute.	Head, h, in feet.	Power at machine terminals, in kilowatts.	Discharge, in cubic feet per second.	Performance on 30-ft. Head.				Over-all efficiency on Unit No. 13, in percentage.	Mean temperature of water, in degrees, centigrade.	Weight of water, in pounds per cubic foot.	Remarks
			On reg. cyl.	Actual.					Speed, in revolutions per minute.	General efficiency = 100%. Power at mach. ter., in kilowatts.	General efficiency = 100%. Power at mach. ter., in horse-power.	Discharge, in cubic feet per second.				
1914																
9-2	46	25	13½	5.0	102	31.30	3 239	1 466	99.9	3 039	4 074	1 435	83.52	20.2	62.315	Dosed tail sampling pipes on upper section. Tests 26-46, both inclusive. Dosed tail sampling pipes on lower section. Tests 22-23. Eight samples also taken on upper section, Tests 22-23.
8-22	23	30	13¾	5.12	98.0	28.76	2 912	1 431	100.0	3 102	4 158	1 461	83.70	18.5	62.337	
8-27	33	30	13⅞	5.18	101.0	31.18	3 289	1 491	99.1	3 104	4 161	1 463	83.65	18.7	62.335	
9-1	44	25	13 15/16	5.20	101.0	30.77	3 262	1 497	99.8	3 140	4 209	1 479	83.71	19.6	62.323	This unit generates direct current. Gate locked on all tests. Discharge obtained by method of unbalanced evaporations. All samples titrated separately. Dosing pipes not plugged in middle.
8-31	41	30	14⅛	5.30	102.0	31.46	3 387	1 520	99.6	3 154	4 228	1 484	83.80	18.8	62.334	
8-26	26	30	14	5.24	100.0	29.60	3 102	1 482	100.7	3 165	4 243	1 492	83.63	18.4	62.338	
8-31	42	25	14¼	5.36	103.0	31.88	3 473	1 542	100.0	3 170	4 249	1 496	83.54	19.0	62.331	
8-21	22	30	14¼	5.36	99.7	29.72	3 148	1 496	100.0	3 192	4 279	1 503	83.74	19.0	62.331	
8-26	32	30	14½	5.48	101.0	31.41	3 442	1 552	98.7	3 213	4 307	1 517	83.50	18.7	62.335	
8-28	39	30	14¼	5.36	99.5	29.07	3 068	1 495	101.0	3 216	4 311	1 518	83.52	18.6	62.336	Head computed. Part of record lost.
9-2	45	25	14½	5.48	103.3	32.50	3 631	1 582	99.4	3 220	4 316	1 520	83.54	20.2	62.315	
9-1	43	25	14¾	5.60	99.5	29.76	3 219	1 536	100.0	3 258	4 367	1 542	83.30	19.4	62.326	
8-28	40	30	15	5.72	98.0	28.76	3 099	1 532	100.0	3 301	4 425	1 564	83.22	19.0	62.331	Pen on ammeter caught.

TABLE 57.—FINAL COMPUTATIONS. TESTS OF UNIT NO. 12. TESTS 200-212, INCLUSIVE. SEPTEMBER 28TH–OCTOBER 13TH, 1914.

Date.	Test.	Duration, in minutes.	Gate Opening, in Inches.		Speed, in revolutions per minute.	Head, h, in feet.	Net power at machine terminals, in kilowatts.	Discharge, in cubic feet per second.	Performance on 30-Ft. Head.				Over-all efficiency on Unit No. 12, in percentage.	Mean temperature of water, in degrees, centigrade.	Weight of water, in pounds per cubic foot.	Remarks.
			On reg. cyl.	Actual.					Speed, in revolutions per minute.	General Efficiency, = 100%. Power at machine terminals, in kilowatts.	General Efficiency, = 100%. Power at machine terminals, in horse-power.	Discharge, in cubic feet per second.				
1914. 9-30	202	28	13¼	5.20	101.7	30.17	3 082	1 476	101.4	3 056	4 097	1 471	81.86	15.6	62.368	All samples except on Test 201 are titrated separately. Dosing pipes plugged in middle. Dosed tail sampling pipes on upper section. These tests are not as satisfactory as the third series on Unit No. 13, there being more irregularity in tail-water samples. The discharge was obtained by the methods of Unbalanced Evaporations and Special Dilutions, the average of the two being used. This unit generates direct current. On Tests 200 to 204, inclusive, sample pipe No. 27 was broken, sample No. 27 actually came from top of upper draft-tube. Value for this sample interpolated. Pipe repaired after Test 204. Dosed tail samples averaged chemically in pairs, on Test 201, No. 43 with No. 51, No. 29 with No. 31, No. 55 with No. 41, No. 25 with No. 42, No. 28 with No. 30, No. 22 with No. 44. Gate locked on all tests. Rain during Tests 201 and 211.
10-6	206	30	13⅝	5.26	100.0	30.55	3 207	1 492	99.1	3 121	4 183	1 479	83.12	13.3	62.389	
10-7	207	34	13⅝	5.26	102.0	30.56	3 179	1 487	101.1	3 092	4 145	1 473	82.71	13.27	62.389	
10-13	212	30	13⅝	5.26	101.5	30.41	3 160	1 491	100.8	3 096	4 150	1 481	82.36	13.55	62.387	
9-29	201	30	13¾	5.32	102.0	31.05	3 336	1 515	100.3	3 168	4 246	1 490	83.76	16.1	62.363	
10-2	204	30	13¾	5.32	100.3	30.92	3 287	1 505	98.8	3 141	4 210	1 482	83.49	15.2	62.372	
10-8	208	25	13¾	5.32	100.5	30.24	3 160	1 499	100.1	3 123	4 186	1 493	82.41	14.0	62.383	
10-12	211	25	13¾	5.32	103.0	31.85	3 389	1 545	100.0	3 098	4 153	1 499	81.43	14.0	62.383	
10-5	205	30	13⅞	5.38	101.6	30.21	3 181	1 512	101.1	3 148	4 220	1 506	82.36	14.25	62.381	
10-9	209	25	13⅞	5.38	100.2	31.12	3 330	1 561	98.4	3 152	4 225	1 532	81.05	14.1	62.382	
10-10	210	25	13⅞	5.38	102.4	31.26	3 377	1 541	100.3	3 174	4 255	1 510	82.82	14.5	62.379	
10-1	203	25	14	5.44	99.5	30.62	3 275	1 519	98.5	3 176	4 257	1 504	83.19	15.3	62.371	
9-28	200	25	14¼	5.56	101.0	30.65	3 341	1 568	99.9	3 235	4 337	1 551	82.19	16.05	62.364	
														Ave.	62.378	(Used on this series of tests.)

TABLE 58.—Final Computations. Tests of Unit No. 11. Tests 105-109, Inclusive. First Series.
September 12th–September 18th, 1914.
Tests 101-104, Inclusive, are Distribution Tests.

Date.	Test.	Duration, in minutes.	Gate Opening, in Inches. On reg. cyl.	Gate Opening, in Inches. Actual.	Speed, in revolutions per minute.	Head, h, in feet.	Net power at machine terminals, in kilowatts.	Discharge, in cubic feet per second.	Performance on 30-Ft. Head. Speed, in revolutions per minute.	Performance on 30-Ft. Head. General Efficiency = 100%. Power at machine terminals, in kilowatts.	Performance on 30-Ft. Head. General Efficiency = 100%. Power at machine terminals, in horse-power.	Performance on 30-Ft. Head. Discharge, in cubic feet per second.	Over-all efficiency on Unit No. 11, in percentage.	Mean temperature of water, in degrees, centigrade.	Weight of water, in pounds per cubic foot.	Remarks.
1914 9-12	105	15	13.92	5.10	100.2	30.36	3 218	1 446	99.6	3 161	4 237	1 437	86.70	17.0	62.354	Unit on governor. Water rheostat connected on this test only.
9-18	109	20	14.0	5.14	102.7	31.23	3 293	1 487	100.7	3 101	4 157	1 458	83.84	18.0	62.343	
9-17	108	20	14¼	5.26	103.5	30.98	3 280	1 482	101.8	3 125	4 189	1 458	84.48	17.8	62.346	Dredge started during test. Head varied.
9-15	106	20	14⅛	5.38	103.4	31.99	3 561	1 539	100.1	3 233	4 334	1 490	85.53	17.2	62.352	Valves closed on Dosing Pipes Nos. 3, 4, 9, and 10, because Pipes 3 and 4 broke just before starting test.
9-16	107	20	14¾	5.50	103.0	30.78	3 388	1 519	101.7	3 260	4 370	1 500	85.66	17.6	62.348	
														Ave.=	62.349	

Notes.—All samples titrated separately. Dosing pipes plugged in middle. No excitation power readings taken. Dosed tail sampling pipes on upper section. Discharge obtained by two methods: Unbalanced Evaporations and Special Dilutions. Average of the two results used. This unit generates alternating current. Gates locked on all tests except 105.

TABLE 59.—FINAL COMPUTATIONS. TESTS OF UNIT NO. 11. TESTS 111-112, INCLUSIVE. SECOND SERIES.
NOVEMBER 9TH–NOVEMBER 21ST, 1914.
No Test Run on No. 110.

Date.	Test.	Duration, in minutes.	Gate Opening, in Inches.		Speed, in revolutions per minute.	Head, h, in feet.	Net power at machine terminals, in kilowatts.	Discharge, in cubic feet per second.	Performance on 30 Ft. Head.				Over-all efficiency on Unit No. 11, in percentage.	Mean temperature of water, in degrees, centigrade.	Weight of water, in pounds per cubic foot.	Remarks.
			On reg. cyl.	Actual.					Speed, in revolutions per minute.	General efficiency, = 100 %. Power at machine terminals, in kilowatts.	General efficiency, = 100 %. Power at machine terminals, in horse-power.	Discharge, in cubic feet per second.				
1914.																
11-10	112	20	14⅜	5.32	103.5	30.51	3 161	1 488	102.6	3 082	4 131	1 475	82.25	8.0	62.421	All samples titrated separately. Dosing pipes plugged in middle. Dosed tail sampling pipes on upper section.
11-11	113	20	14⅜	5.32	103.0	31.84	3 413	1 513	100.0	3 121	4 184	1 469	83.65	8.0	62.421	Discharge obtained by special dilution method. This unit generates alternating current.
11-12	114	20	14⅝	5.44	102.0	30.88	3 312	1 511	100.5	3 171	4 251	1 489	83.85	8.0	62.421	Gate locked on all tests.
11-9	111	20	14⅝	5.44	106.9	31.36	......	1 525	104.6	......	......	1 492	.... .	7.7	62.422	No power readings taken.
11-18	119	20	14¾	5.50	105.5	30.99	3 339	1 531	103.8	3 180	4 263	1 507	83.08	5.0	62.428	
11-13	115	20	14⅞	5.56	103.5	31.31	3 419	1 536	101.3	3 206	4 298	1 504	83.93	7.8	62.422	
11-14	116	20	15	5.62	104.0	32.24	3 607	1 572	100.3	3 237	4 339	1 516	84.06	6.4	62.426	Rained hard during test.
11-20	121	20	15⅛	5.68	103.5	31.28	3 483	1 580	101.4	3 271	4 385	1 547	83.25	4.8	62.428	Snowed during test.
11-17	118	20	15¼	5.74	103.7	33.56	3 889	1 609	98.0	3 287	4 406	1 521	85.08	6.0	62.427	
11-21	122	20	15¼	5.74	103.8	31.67	3 581	1 563	101.0	3 302	4 426	1 521	85.46	4.2	62.428	Tail-race samples spoiled.
11-16	117	20	15½	5.86	105.5	32.90	3 829	1 639	100.7	3 334	4 469	1 565	83.87	7.0	62.424	
11-19	120	20	15¾	5.98	105.5	32.28	3 784	1 664	101.7	3 390	4 544	1 605	83.15	5.2	62.428	
														Ave.=	62.425	(Used on this series of tests.)

tions of the tests were not so uniform nor so satisfactory, from the standpoint of turbine driving. The efficiency is markedly less for this unit than for Unit No. 13, and this may be attributed to the relatively higher degree of turbulence of the head-water. It can scarcely be doubted that turbulence in the head-water has an effect on the efficiency of a water-wheel, though this would probably be small, relatively. Tests to ascertain the truth or falsity of this proposition are highly desirable.

Table 58, Unit No. 11, First Series.—These tests were started on September 12th, 1914, but it was found necessary to shut down for repairs on September 17th, before there had been time to get the equipment adjusted. Test 105 is wholly untrustworthy, as the governor was not blocked and the gate varied over a large range. The power was absorbed by a large water-rheostat, but this was improperly placed in the tail-water, which varied in elevation from moment to moment, thus changing the load and causing the variations of gate mentioned. The load was also affected by variations of salt content in the tail-water. The rheostat was not used in any test after the first one, Test 105.

The writer is not an advocate of rheostat tests for efficiency trials of hydro-electric units in place. It is very true that a uniform load can be secured in this way, but it is far better to secure tests under actual operating conditions. That such tests can be accurately conducted on either direct or alternating circuits is amply proved by the whole series which form the subject of this paper. Moreover, it is demonstrated herein that considerable variations of load do not affect seriously the results of a test which is properly conducted, especially if such variations are restricted within certain limits. One such test will check another under similar conditions of equal average load with surprising exactness.

Table 59, Unit No. 11, Second and Last Series.—These tests are more satisfactory than those on Unit No. 12, but not so good as those on Unit No. 13. When corrected for speed variations, the results agree very consistently, except in the case of Test 122, which must be discarded entirely. In preparing for this test it was found that all the available sample bottles were full from other tests. Accordingly, some bottles which had been set aside with salt solution were rinsed out and used, but the tail-water samples show conclusively that the bottles were

not sufficiently cleansed. Some of the samples titrated nearly twice as high as they should. Certain samples, however, had the appearance of being about right in respect to their salt content, and the test has been worked up on this basis with the result that the final figures are not far from the truth, but the facts rob the result of any intrinsic value. Test 118 seems to agree with Test 122, but an inspection will show that there was considerable difference (3 rev. per min.) in speed. The tests, therefore, are not directly comparable.

110.—Plottings of the Tests.—The final adjustment of the tests for the purpose of determining turbine efficiencies cannot now be made, as the efficiencies of the generators are not known for the various conditions of performance. The over-all efficiencies of the units, however, would appear to be well determined by the curves which may be drawn with reference to the final computations, Section 109.

Plate LX, Unit No. 13, First Series.—These were the lettered tests in which the power was measured by the switch-board meters only. Although the discharge measurements were made with considerable accuracy, there is a strong probability that some carelessness in the laboratory resulted in the interchange of salt-solution samples for at least one pair, and possibly two pairs of tests. The most questionable pair in this respect is *T*, *U*, where an interchange of salt-solution samples would make better harmony as regards results. The fact which lends weight to this view is that at about this time the salt-solution samples were, on several occasions, titrated on the morning following the titration of the other samples for the same test—a procedure which is strongly to be condemned. Tests *T* and *U* are subject to this suspicion. They would be far more harmonious if the salt-solution titrations were interchanged in the calculations, as may be seen by an inspection of the data and results.

The power on these lettered tests may be rendered comparable with the later series by applying a correction to the switch-board instruments.

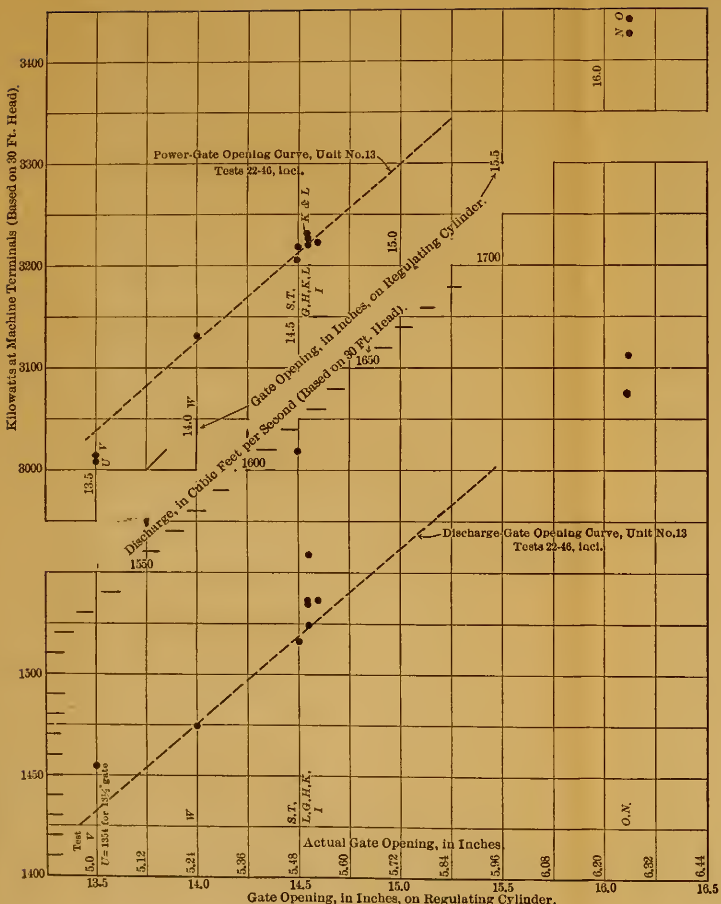
Plate LXI, Unit No. 13, Second Series.—In this series, computed in Table 55, Section 109, the tail-race samples were taken principally from the lower sampling section. It has been mentioned that some leaks were discovered in the risers for these samples at points where contamination would be likely to occur. Hence, in computing Table

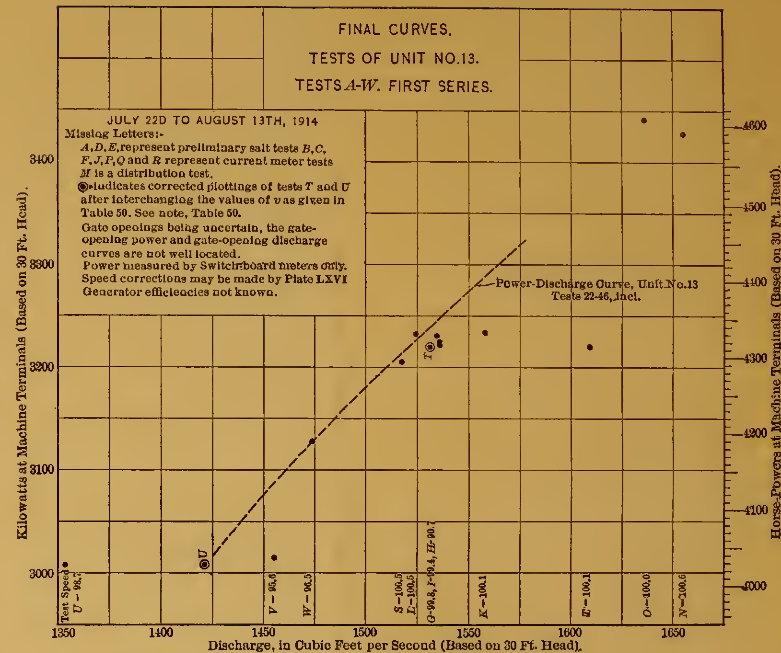
55, the titrations of the samples which appeared to be too low were at first corrected by cutting them out and inserting the averages of some of the adjacent samples in their places. The next diagram gives a plotting of the tests without any arbitrary corrections, and these check the third series even better than the plottings of the diagram now in question.

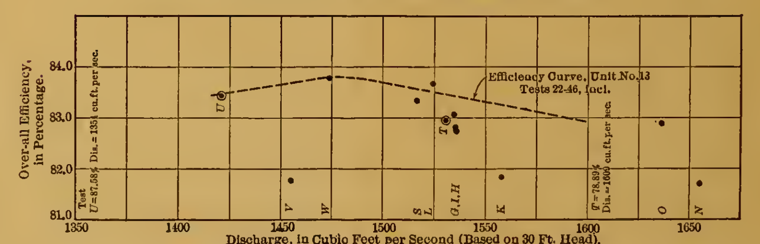
Tests 6, 9, 10, 11, 12, and 19 are the only ones of this series in which the tail-water samples were titrated in complete detail. In all the remaining tests the twelve regular tail-water samples were combined in pairs, by mixing equal parts of each one of every pair of samples, thus reducing the number of titrations for these samples from twelve to six. The small number of samples, however, is not the worst feature. The main trouble is that, with so few samples, it is not possible to correct for non-uniformity of distribution with any degree of certainty.

It is also to be observed that these particular tests were intended to cover a larger range of speed variations than was permissible under the terms of the contract. It was hoped by the writer that enough material could be accumulated in this way to diagram the wheels. Unfortunately, the representatives of the Allis-Chalmers Company did not agree unreservedly to such a plan, and it was necessary, therefore, to confine the work as closely as possible to the contract conditions of 100 rev. per min. On account of these speed variations, coupled with the uncertainties as to distribution of salt in the tail-race under such conditions, especially in the case of the partial tests just mentioned, it must not be expected that the points for the tests will plot in a smooth curve.

Test 19 of this series, and Tests 22 and 23 of the next, or third, series are worthy of particular notice, as the full quota of twelve samples was taken at the lower section while eight well-distributed samples were taken in the upper section, about 8 ft. farther up stream, and well away from any influence of unsalted tail-water. These were the last tests for which any samples were taken in the lower section. This was on account of the difficulty of maintaining the sampling system there, so as to prevent leaks in the pipes. Tests 20 and 21 were merely partial tests, as explained. After cutting out one sample on the lower section in Test 22 and three in Test 23, on account of breaks in the correspond-


Kilowatts at Machine Terminals (Based on 30 Ft. Head).
Power-Gate Opening Curve, Unit No.13
Tests 22-46, incl.
Gate Opening, in Inches, on Regulating Cylinder.
Discharge, in Cubic Feet per Second (Based on 30 Ft. Head).
Discharge-Gate Opening Curve, Unit No.13
Tests 22-46, incl.
Actual Gate Opening, in Inches
Gate Opening, in Inches, on Regulating Cylinder.
U = 1354 for 13½ gate


FINAL CURVES.
TESTS OF UNIT NO.13.
TESTS A-W. FIRST SERIES.
JULY 22D TO AUGUST 13TH, 1914
Missing Letters:-
A,D,E, represent preliminary salt tests B,C,
F,J,P,Q and R represent current meter tests
M is a distribution test.
◉ indicates corrected plottings of tests T and U
after interchanging the values of v as given in
Table 50. See note, Table 50.
Gate openings being uncertain, the gate-
opening power and gate-opening discharge
curves are not well located.
Power measured by Switchboard meters only.
Speed corrections may be made by Plate LXVI
Generator efficiencies not known.
Power-Discharge Curve, Unit No.13
Tests 22-46, incl.
Kilowatts at Machine Terminals (Based on 30 Ft. Head).
Horse-Powers at Machine Terminals (Based on 30 Ft. Head).
Discharge, in Cubic Feet per Second (Based on 30 Ft. Head).


Over-all Efficiency, in Percentage.
Efficiency Curve, Unit No.13
Tests 22-46, incl.
Discharge, in Cubic Feet per Second (Based on 30 Ft. Head).

ing pipes, Table 60 was compiled. It shows an interesting comparison of titrations of the two sets of samples in these tests.

TABLE 60.—COMPARISON OF THE AVERAGE TITRATIONS FROM THE UPPER SAMPLING SECTION WITH THE AVERAGE FROM THE LOWER SECTION.

(The averages are from eight samples on the upper section and twelve on the lower, except that one sample on the lower section for Test 22, and three for Test 23, have been suppressed on account of leaks in the corresponding pipes.)

TEST 19.

	UPPER SECTION.		LOWER SECTION.	
	W.	E.	W.	E.
Upper half	58.41	58.07	58.22	58.30
Lower half	58.55	59.08	58.71	59.12
Means	58.48	58.58	58.46	58.71

TEST 22.

Upper half	59.45	59.52	59.05	58.66
Lower half	57.65	57.90	58.20	58.10
Means	58.55	58.71	58.62	58.38

TEST 23.

Upper half	60.89	60.72	60.26	60.12
Lower half	57.06	56.95	56.52	58.10
Means	58.98	58.84	58.39	59.11

SUMMARY OF MEANS.

Test.	Average upper section.	Average lower section.
19	58.53	58.58
22	58.63	58.50
23	58.91	58.75
Mean	58.69	58.61

From Table 60, therefore, it is apparent that there was no material difference between the concentrations from the upper and lower stations when the pipes on the lower station were in good condition. The small deficit of 0.08 c.c. in nearly 60.00 c.c. may be due to some slight leaks on the lower section which were not discovered, or it may be due to unavoidable errors in the work.

The tests of this series will appear to be more harmonious when compared with the Holyoke tests by using Plate LXVI.

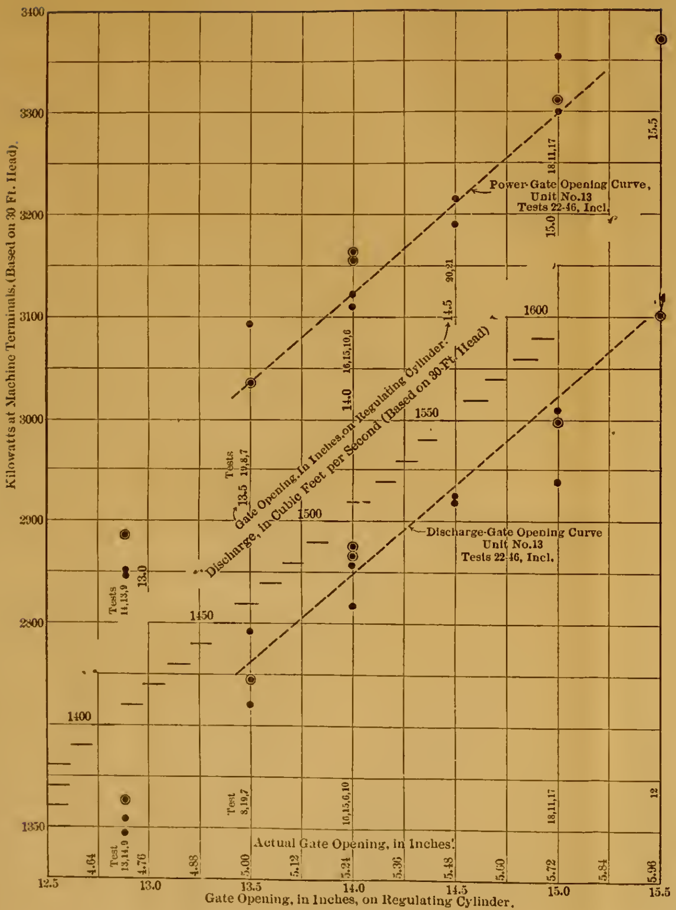
Tests 8, 16, 17, and 18 are very doubtful as plotted. They were off-speed tests, but the principal cause of deviation from correct plotting is probably due to lack of data as to distribution of salt and relative weights. This applies to all the partial tests.

Plate LXII, Unit No. 13, Second Series, Second Plotting.—After plotting this series, as on Plate LXI, where certain tail-water samples were eliminated owing to supposed leaks, the tests were re-plotted, taking into account all the titrations as usual, without regard to the possibility of leaks. This plotting is shown on Plate LXII, and it will be seen that the changes in main are small, but that the final plotting checks that of the third series even better than that of Plate LXI.

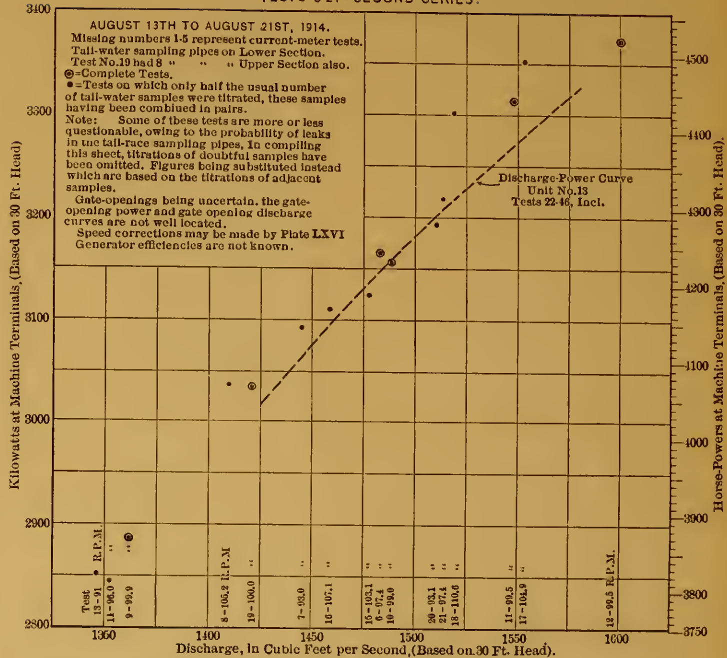
Plate LXIII, Unit No. 13, Third Series.—This series is remarkable for its self-consistency and for the high degree of precision attained in so far as accidental errors are concerned. It has also been shown in Part II that there cannot be any serious systematic error in this series. In all probability, they are the most precise discharge measurements ever made on a hydro-electric unit of large capacity for water.

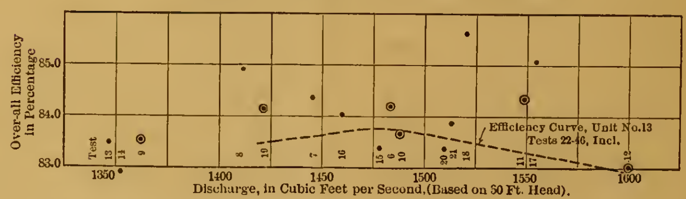
Test 39 is the only one about which there is any question. Unfortunately, during this test, the greatest portion of the head-water observations was lost, thus taking away any opportunity for arriving at the correct head based on observation. The head has been interpolated so as to harmonize the test with the others. Had these observations been lost on any other test, the matter would not have been so serious, because, in Test 39, the head varied by 1½ ft. or more, and it was desirable to have the results of a test, under such conditions, but the only chance in this series was lost.

The close agreement between concentrations on the upper and lower sampling sections in Tests 22 and 23 has been shown in Table 60 in connection with Plate LXI.



FINAL CURVES.
TESTS OF UNIT NO.13
TESTS 6-21 - SECOND SERIES.





Gate Openings.—No effort was made, in any of the tests, to secure accurate records of gate opening. The gates were calipered in the usual manner, and a curve connecting the actual opening with gate-register indications was drawn. These openings, as well as the corresponding gate-register readings, are plotted at the left of the plate, as in the preceding cases.

The method of setting the gate was always to open it from a relatively small opening to the one at which the test was to be run, thus avoiding the effects of back-lash as far as possible. It was quite possible, however, for the gate to move after being set, and it is very clear, from the diagram, that such motions did occur, and always in one and the same direction from the gate-power or gate-discharge curve. This would be the natural consequence of the method of setting the gate, and the consistency of the plottings in this respect adds materially to the weight of the statements concerning the accuracy of this series of tests.

The real precision of the tests comes into view when power and discharge are plotted against one another. An extremely accurate efficiency curve results by computation from this discharge-power curve, and it is seen that, of the plottings of the efficiencies from Table 56, only two or three of the thirteen fall away from the curve by as much as 0.1 per cent.

Plate LXIV, Unit No. 12.—This is the only series of tests on Unit No. 13. The fact that accidental errors are larger than for Unit No. 13 209 and 211 are the only ones plotting seriously wild. Test 211 seems to be low on power, and 209 is high on discharge. Otherwise the tests leave no doubt as to the fact that Unit No. 12 is not so efficient as Unit No. 13. The fact that accidental errors are larger than for Unit No. 13 is due to the greater haste with which the tests were made and to several changes in the personnel of observers, the new men not being so well trained in their work. This is especially true of the power, which, however, is quite consistent with the Allis-Chalmers readings, excepting for Tests 211 and 212. In these two the Allis-Chalmers instruments are inconsistent with both the standard and the switch-board instruments, as the ratios in Table 61, computed by slide-rule, indicate.

The Leeds and Northrup and switch-board readings being the more consistent, it would scarcely seem that the power can be seriously in

error. It is more probable that the wild plottings result from an accumulation of small errors due to one cause or another. A further adjustment of the tests might be made, but this would not make any material change commensurate with the work necessary to effect a satisfactory adjustment.

TABLE 61.—COMPARISON OF THE RATIOS OF POWER READINGS FOR THE SWITCH-BOARD, ALLIS-CHALMERS, AND LEEDS AND NORTHRUP, DIRECT-CURRENT INSTRUMENTS IN THE TESTS ON UNIT NO. 12.

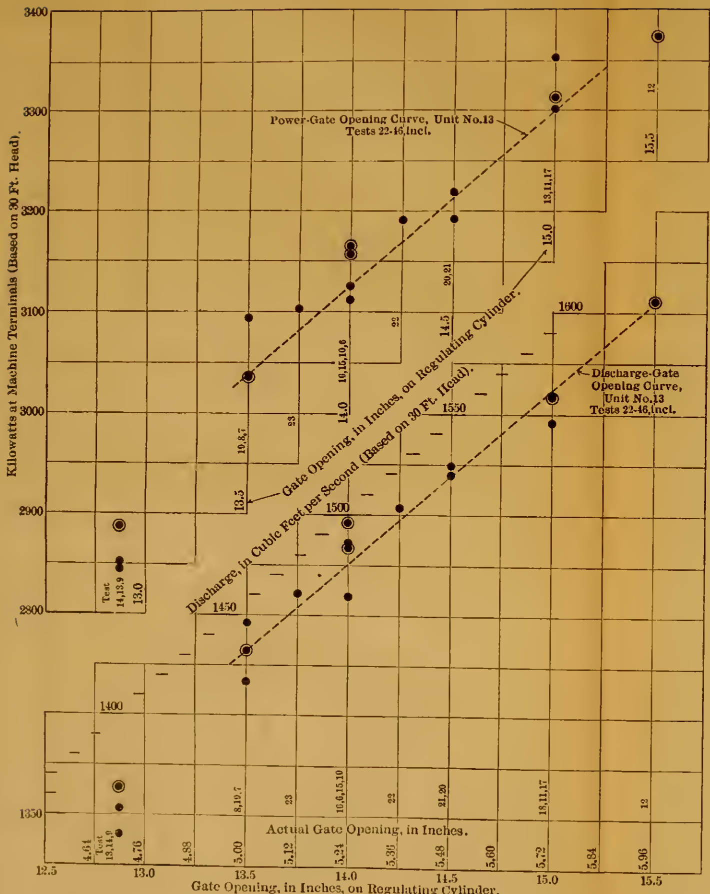
Test No.	Leeds and Northrup / Switch-board	Leeds and Northrup / Allis-Chalmers	Allis-Chalmers / Switch-board
200	1.016	1.001	1.014
201	1.021	1.001	1.020
202	1.026	1.000	1.026
203	1.019	1.003	1.019
204	1.023	1.002	1.021
205	1.030	1.004	1.026
206	1.023	1.002	1.021
207	1.028	1.002	1.026
208	1.030	0.999	1.031
209	1.027	1.004	1.026
210	1.030	1.003	1.026
211	1.021	0.989	1.033
212	1.025	0.991	1.034

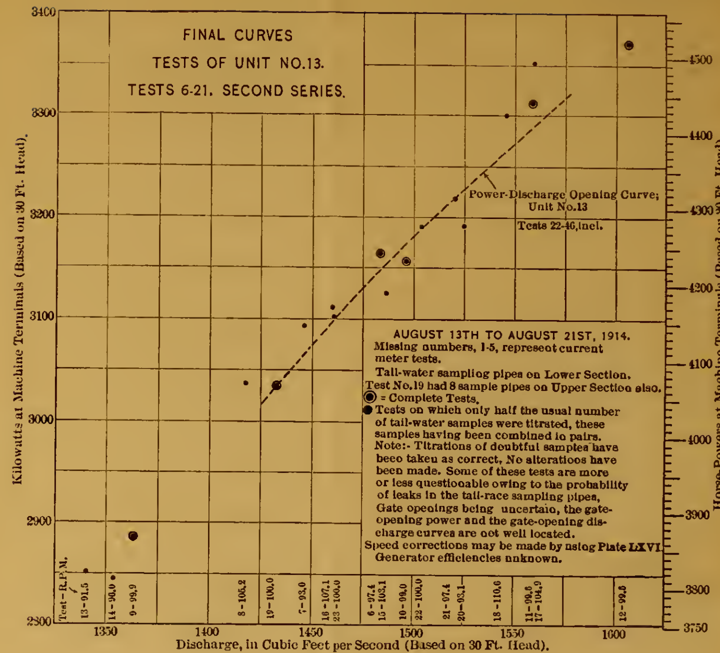
A more satisfactory procedure will be to select those tests which appear to possess the elements of precision. If several of the tests plot on both the gate-discharge and gate-power curves, and are otherwise consistent, they may well be given the greater weight in drawing the final efficiency curve, which, owing to the paucity of tests over a large range of gate openings for this unit, may be extended parallel to the efficiency curve of Unit No. 13, which has been drawn on the diagram as a broken line.

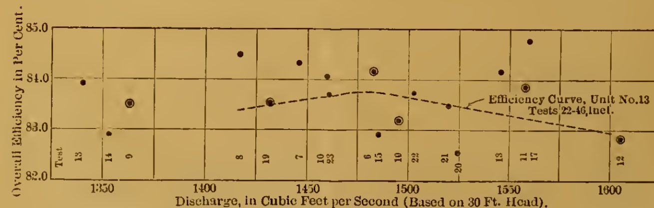
Tests 207 and 212 plot well in this respect, as do 200, 205, and 208. Several others, 202 and 210, for example, are not seriously off, in this regard, though a combination of discrepancies throws the former downward.

It is not difficult to account for a slight deficit in the performance of Unit No. 12 as compared with Unit No. 13. The hydraulic conditions were plainly not so good for the former. Indeed, the turbulence in the head-races of the units increases in violence as the west end of

PLATE LXII.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1915.
GROAT ON
CHEMI-HYDROMETRY.







the power-house is approached, No. 11 being in a less favorable location than No. 12. There is also a possibility that No. 12 was not so well adjusted as No. 13, very small differences in this respect having an appreciable effect on the net performance of the unit. This, however, was contrary to the expectations of the makers, who claimed that Unit No. 12 was in better condition than Unit No. 13.

The probability of a constant error as large as 0.75% in the tests of No. 12 is very small, as has been amply shown by a full discussion of the errors of the chemical method in Part II. There may be a possibility of 0.1 or 0.2% error of this class, but, without further evidence, there is no justification for such an assumption.

Plate LXV, Unit No. 11.—The plotting of the first series of tests, 105-109, on this unit is shown by circles on Plate LXV. The discharge measurements are fairly consistent, but the power is not. Test 105 is wholly untrustworthy as to power, as the governor was free and the power absorbed by the water rheostat, which was improperly connected and hung in the tail-race, was affected by variations in water level and variations of salt concentration. Consequently, the gate varied throughout a large range, thus throwing considerable doubt on this particular test. All the other tests were made on the regular load, and an attempt was made to keep the power factor in the vicinity of 85. Table 62 gives the power factor as computed and also as read from the power-factor indicator.

TABLE 62.—Power Factor Observations in Tests on Unit No. 11. Second Series.

Test No.	Power factor computed.	Power factor indicated.
111	.....	.....
112	0.855	.....
113	0.814	.....
114	0.835	.....
115	0.851	0.845
116	0.833	0.868
117	0.862	0.950
118	0.843	0.857
119	0.867	0.920
120	0.865	0.950
121	0.852	0.860
122	0.864	0.860
Mean	0.849	0.889
Mean, excluding 112	0.853	

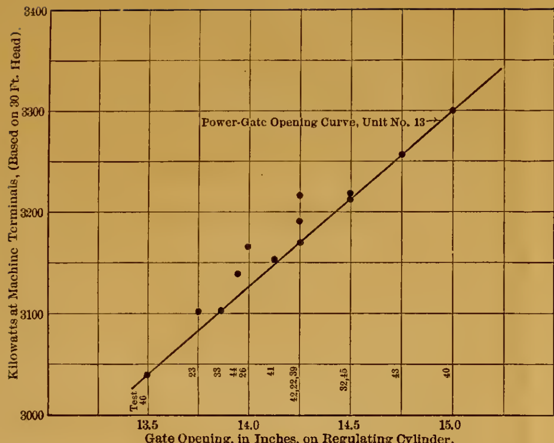
The average power factor, therefore, was almost exactly 0.85, which was the contract requirement.

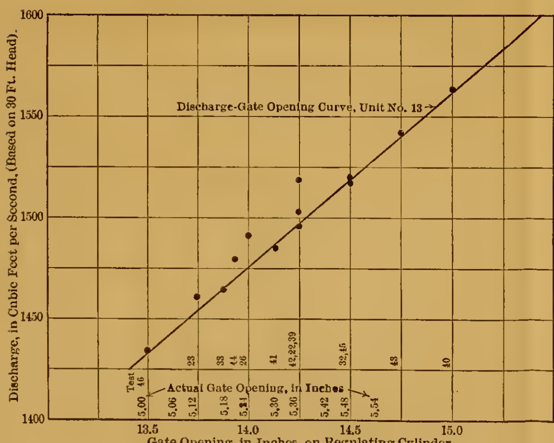
All the tests of the second series have a very consistent relation to the Holyoke tests on the model runner when stepped up to the full size, as on Plate LXVI, with the exception of Test 122, which cannot be considered as trustworthy on account of the errors introduced into the tail-race samples by reason of a change to sample bottles which were not thoroughly cleaned, as mentioned in Section 109. Where there are divergencies from a satisfactory plotting on Plate LXV, there are also properly related divergencies of speed from the contract requirement of 100 rev. per min. On the whole, the second series of tests on Unit No. 11 approximates in precision the third series on Unit No. 13, with only such errors as may be expected with the less reliable power readings and the greater haste in making the tests on the former. The chemical work is not quite as satisfactory as on Unit No. 13, but shows no serious evidence of rough work.

The fact that the efficiency does not show as high, relatively, as that of the direct-current machines is probably due to the fact (mentioned in connection with the tests on Unit No. 12) that the hydraulic conditions grow steadily worse as this end of the power-house is approached from the east, where Units Nos. 13, 14, and 15 are situated. On the whole, there is no evidence that there is any serious constant error in respect to the finally determined efficiency curve for Unit No. 11.

111.—Diagram of Turbine Performance.—It is only very recently that systematic attempts have been made to secure greater efficiency of power-house operation. This is in line with suggestions made in reports by the writer in 1910 and subsequently. As a general rule, each power development will require individual investigation to secure a complete answer to this question. There is one feature of it, however, which can be treated generally, and that is the characteristics of the hydro-electric units in the power-house. The performance of these units is subject to exact analysis, and diagrams of performance can be constructed which will facilitate an intelligent and effective oversight of this operation.

Most of the diagrams which have been constructed for the analysis of turbine performance have been made by designers and builders, and such diagrams, as a rule, are not adapted to the uses of power-house

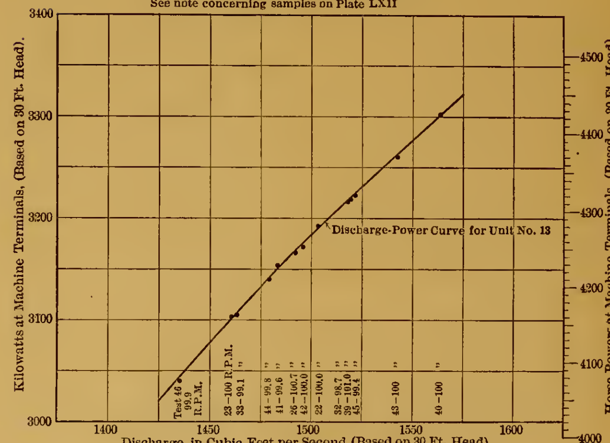


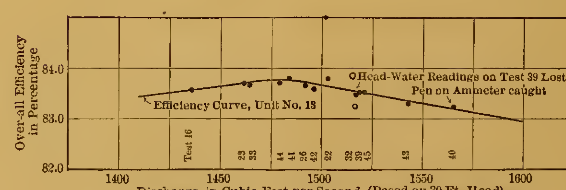


FINAL CURVES.
TESTS OF UNIT NO. 13,
TESTS 22-46. THIRD SERIES.
AUGUST 21ST TO SEPTEMBER 2D, 1914.

Tests 24, 25, 27-31, and 34-38 represent current-meter tests and are not shown on this plate. Dosed Tail Sampling Pipes on Upper Section, Tests 26-46. Dosed Tail Sampling Pipes on Lower Section, Tests 22-23. Tests 22 and 23 had 8 Sampling Pipes on Upper Section also. The speed in this series of tests is so near to 100 Rev. per Min. that no correction in plotting has been made, except in Tests 32 and 39. o = Plotting corrected to 100 Rev. per Min. by Plate LXVI. Gate openings being uncertain, the gate-opening power and gate-opening discharge curves are not well located.

Generator efficiencies unknown. Test 19, Plate LXII might be added to this series with propriety.
See note concerning samples on Plate LXII





superintendents and overseers. Therefore, it is very desirable to have diagrams constructed especially, and the following method has been used successfully by the writer for a number of years.

Relative to the performance of hydraulic turbines, theory and experience prove the following theorems, which, for brevity, are stated in the form of equations:

$$\text{At any fixed gate opening} \ldots \left\{ \begin{array}{l} N = n\, h^{\frac{1}{2}} \\ P = p\, h^{\frac{3}{2}} \\ Q = q\, h^{\frac{1}{2}} \end{array} \right\} \ldots\ldots\ldots (196)$$

In which, N = number of revolutions per minute;

P = number of horse-powers developed by the runners;

Q = discharge, in cubic feet per second;

h = the head acting upon the wheels, in feet;

and n, p, q = obvious coefficients of performance.

These coefficients have been defined by some writers as being, respectively, the speed, power, and discharge of the turbine when operating on 1 ft. head. That is, the corresponding values of N, P, and Q when $h = 1$ are the values of n, p, and q. This is somewhat misleading, because, under this extreme condition, it might be impossible to get the turbine to operate. At least, it is not likely that such a definition should be taken literally. Coefficient of performance is a better term.

Again, both theory and experience show that p and q depend on the value of n, so that, in addition to Equations (196), we have

$$\text{At any fixed gate opening} \ldots \left\{ \begin{array}{l} p = f_1\,(n) \\ q = f_2\,(n) \end{array} \right\} \ldots\ldots\ldots (197)$$

which symbolize the fact that both p and q are determined as soon as the value of n becomes fixed. Therefore, when n is fixed, both P and Q are definitely connected with h. In words, the theorem may be expressed as follows:

When the gate opening of the turbine and relative peripheral speed of the runner are held constant, that is, when the speed is adjusted to the head, the power of a turbine varies as the square root of the cube of the head, and the discharge varies as the square root of the head.

It is seldom, however, that the gate of a turbine can be set accurately by the ordinary gate mechanism, and, conversely, it is seldom that

the opening can be determined from the reading of a register operated by the mechanism. In any case, it is not necessary to successful power-house operation to know at all times what the gate opening may happen to be. The object may be accomplished in another way. Instead of using gate opening, the discharge or power of the turbines, and therefore of the power-plant, much more important quantities for the power-house superintendent to know, may be made use of with equal facility.

Thus the relations expressed in Equations (197), though they may not be stated in the form of equations, are, nevertheless, equivalent to two equations between the four quantities, n, p, q, and gate opening. Consequently, a single relation may be derived from them connecting only the three quantities, n, p, and q, the fourth having been eliminated. Therefore, it may be formally stated that:

Each of the three quantities, n, p, and q, are dependent on the values of the other two only, the relation being expressible in the form of an isogram.

In particular, p may be expressed as a curved scale (series of curves) drawn to the rectangular co-ordinates given by n as abscissas and q as ordinates without reference to gate opening.

Now, the efficiency of a turbine may be written

$$\phi = \frac{550}{\gamma} \frac{P}{Q\,h} \qquad \text{.......................(198)}$$

from which, by substituting values from Equations (196), we have

$$\phi = \frac{550}{\gamma} \frac{p}{q} \qquad \text{.......................(199)}$$

γ being the specific weight of water.

Therefore, ϕ, like p, may be represented isogrammatically with reference to the same system of co-ordinates, n and q.

It follows from these relations that a diagram can be constructed connecting the speed, head, and speed coefficient, n, according to the relation indicated by the first of Equations (196), resulting in an isogram giving the values of n in the form of a curved scale, where values of h and N are plotted as rectangular co-ordinates. Better still, the curved scale for the speed coefficient becomes a system of radiating straight lines if the square roots of h are taken as the ordinates, or abscissas, as convenience may suggest.

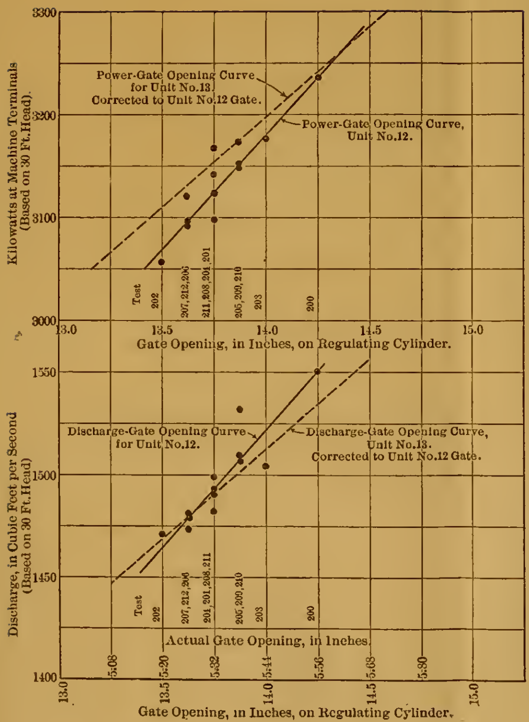
FINAL CURVES,
TESTS OF UNIT NO.12.
SEPTEMBER 28TH TO OCTOBER 13TH, 1914.

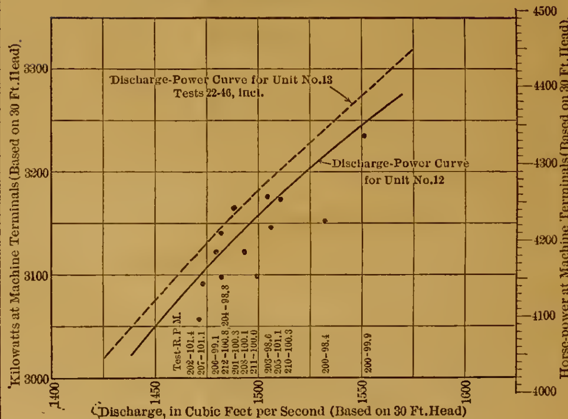
o=Plotting from tests corrected to 100 Rev. per Min. by Plate LXVI

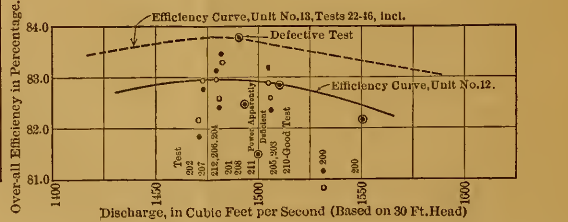
◉ Plotting from tests which do not change by Plate LXVI

Note: Correction of plottings by Plate LXVI uncertain to a small extent, as generator efficiencies are not known exactly.

Gate openings being uncertain, the gate-opening power and gate-opening discharge curves are not well located.







Hence this isogram for n, N, and h, when plotted to the square roots of h, provides a scale of straight lines representing n, and this scale may be taken as the abscissas of a system of co-ordinates in which an arbitrarily chosen scale for q represents the ordinates.

This latter system may then be used in constructing scales for ϕ and p, referred to n and q.

Finally, the original scale for head, h, may be used in conjunction with the scale for power coefficient, p, to form a system of co-ordinates (non-rectangular) for plotting the power of the turbine, P. In similar manner, the head scale and scale for discharge coefficient, q, determine a system of co-ordinates for plotting the discharge, Q, of the turbine.

Plate LXVI illustrates this method of plotting turbine performance. It gives the performance of one of the 80-in. twin-runner hydraulic turbine units which were tested, based on the results of Test No. 2 218 at Holyoke.

The speed and head scales and the sloping scale of relative speeds all appear as straight-line scales. These three scales and the corresponding quantities are connected by the first of Equations (196), the head scale being plotted to the square root of h and the speed scale being graduated according to the relative peripheral speed of the runner, rather than to n itself. This relation is given by the equation:

$$\text{Relative peripheral speed} = \frac{\pi}{60\sqrt{2\,g}} \; \frac{d\,N}{h^{\frac{1}{2}}} \dots\dots\dots\dots (200)$$

where d is the diameter (frequently only nominal) of the runner. If s is the relative peripheral speed, the relation expressed in Equation (200) may be written

$$s = 0.00653\,d\,n \dots\dots\dots\dots\dots\dots\dots (201)$$

which is the equation used in the construction of the sloping scale of the diagram.

Having constructed the scale of relative speeds, or relative peripheral speeds, according to convenience, an arbitrarily chosen scale may be constructed for the discharge coefficient, q. The writer prefers to draw this scale to the left of the lowest number of revolutions per minute likely to be of frequent use. The scale may be composed of vertical parallel lines, conveniently spaced with regard to any desired degree of precision. The graduations for this scale appear along the upper margin of the diagram.

After constructing the discharge coefficient scale, which would be clearer if in a distinctive color, say green, the power coefficient may be plotted, say in carmine, with reference to n, or s, and q, according to the relations in Equations (196). Thus, from the records of the test,

$$\left.\begin{aligned} n &= \frac{N}{h^{\frac{1}{2}}} \\ p &= \frac{P}{h^{\frac{3}{2}}} \\ q &= \frac{Q}{h^{\frac{1}{2}}} \end{aligned}\right\} \quad \text{(See Equations (196))}\ldots\ldots\ldots(202)$$

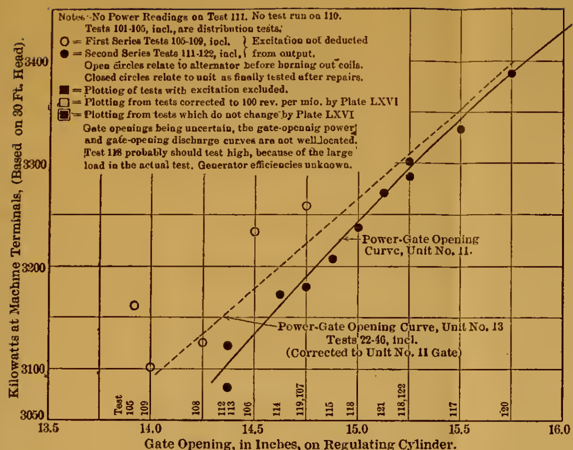
which furnishes the data for plotting p to the co-ordinates, n and q. If it is desired to plot to s and q, then Equations 200, or 201, must be used.

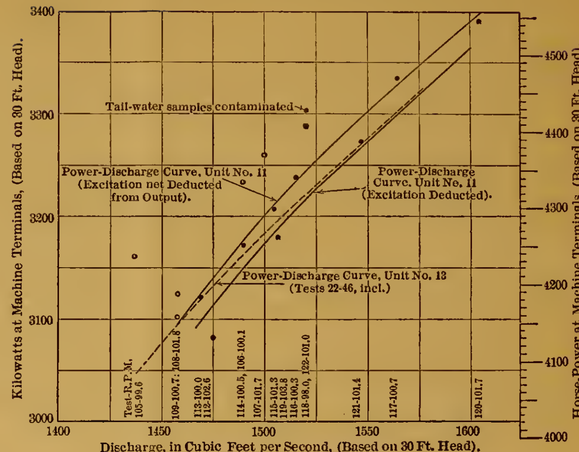
Turning attention now to the co-ordinate scales for p and q, it is seen that each intersects the original scale of heads, produced if necessary. Hence, by the second and third of Equations (196), scales for P and Q may be constructed, as shown on the left half of Plate LXVI. To be in keeping with p and q, the scales for P and Q should be in the same colors as their corresponding coefficients, p and q. Hence, in this case, the scale for Q may be green and that for P may be carmine.

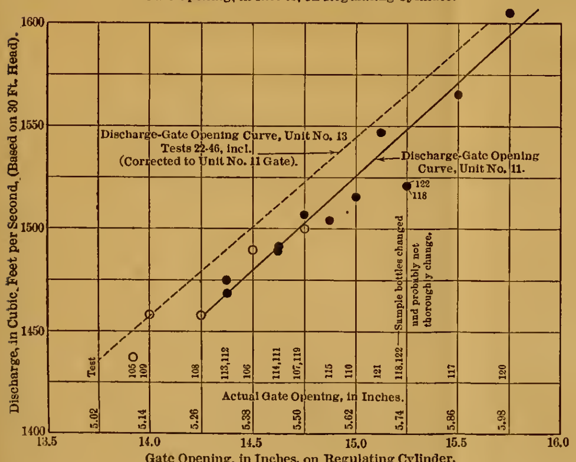
Finally, the efficiency curves may be plotted from the relations of Equations (198) or (199). These curves may be made more distinctive by being plotted in white on a dark background. If desired, curves of gate openings may be plotted in broken, or dotted lines, across the face of the diagram, thus making a complete picture of the operation of the turbine unit in its relation to the actual conditions of head, speed, power, discharge, efficiency, and gate opening.

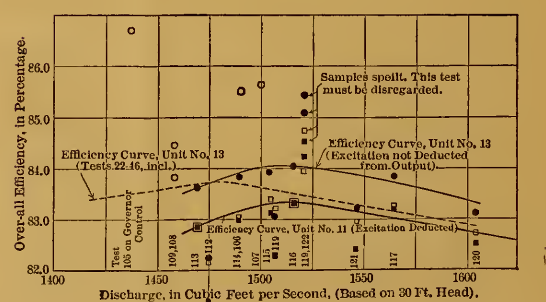
112.—Use of the Turbine Diagram.—Although the plotting of such a diagram involves considerable labor, its use is very simple, and the information it furnishes is extremely valuable in the supervision of power-house operation. The superintendent is able, in a few moments, by examining the diagram, to tell whether a particular unit is operating under favorable conditions or otherwise. He can tell whether a unit is developing all the power it should, under the conditions. If the trash racks become choked, the loss of head makes itself felt by cutting down the output of the unit, and this is instantly shown by

FINAL CURVES.
TESTS OF UNIT NO. 11.
SEPTEMBER 12TH-SEPTEMBER 18TH 1914.
NOVEMBER 9TH-NOVEMBER 21TH 1914.









reference to the diagram with the actual power output, speed, and difference in elevation of head- and tail-water. An inspection of the racks is not nearly so effective as this comparison of the actual and diagrammatic outputs. The racks may be clear but the runners may be choked, perhaps not sufficiently to attract the attention of the operators by a large reduction of power, but the deficit will be clearly shown by the diagram. The superintendent can check the methods of the individual operators by examining the performance of the units at various times.

113.—To Find the Efficiency, Discharge, Relative Speed, and Gate Opening of the Unit, with Given Power, Speed, and Head.—Suppose the head is 30 ft., the speed is 100 rev. per min., and the power is 4 700 h.p. Enter the diagram on the right at 100 rev. per min. on 30 ft. head. The point thus located determines the relative peripheral speed to be about 79½%, which can be read at the extreme left of the diagram near the bottom. It is not necessary to make this reading unless desired. Fix the point just located on the diagram by making a dot, or conveniently with a thumb-tack, or pen-point laid on the diagram. Locate and fix another point at the intersection of 30 ft. head with the curved scale of power at 4 700 h.p. The intersection of the relative speed line through the first fixed point (interpolated by mental estimation when necessary) with the power coefficient line through the second fixed point (also interpolated in this case) locates a third point on the diagram, which, in this case, lies within the closed curve of highest efficiency, 90 per cent. This point should also be fixed on the diagram.

Vertically, above this last point, that is, parallel to the discharge coefficient lines, at the given head, 30 ft. in this case, locate a fourth point which determines the discharge, $Q = 1\,520$ cu. ft. per sec., as shown by the discharge scale.

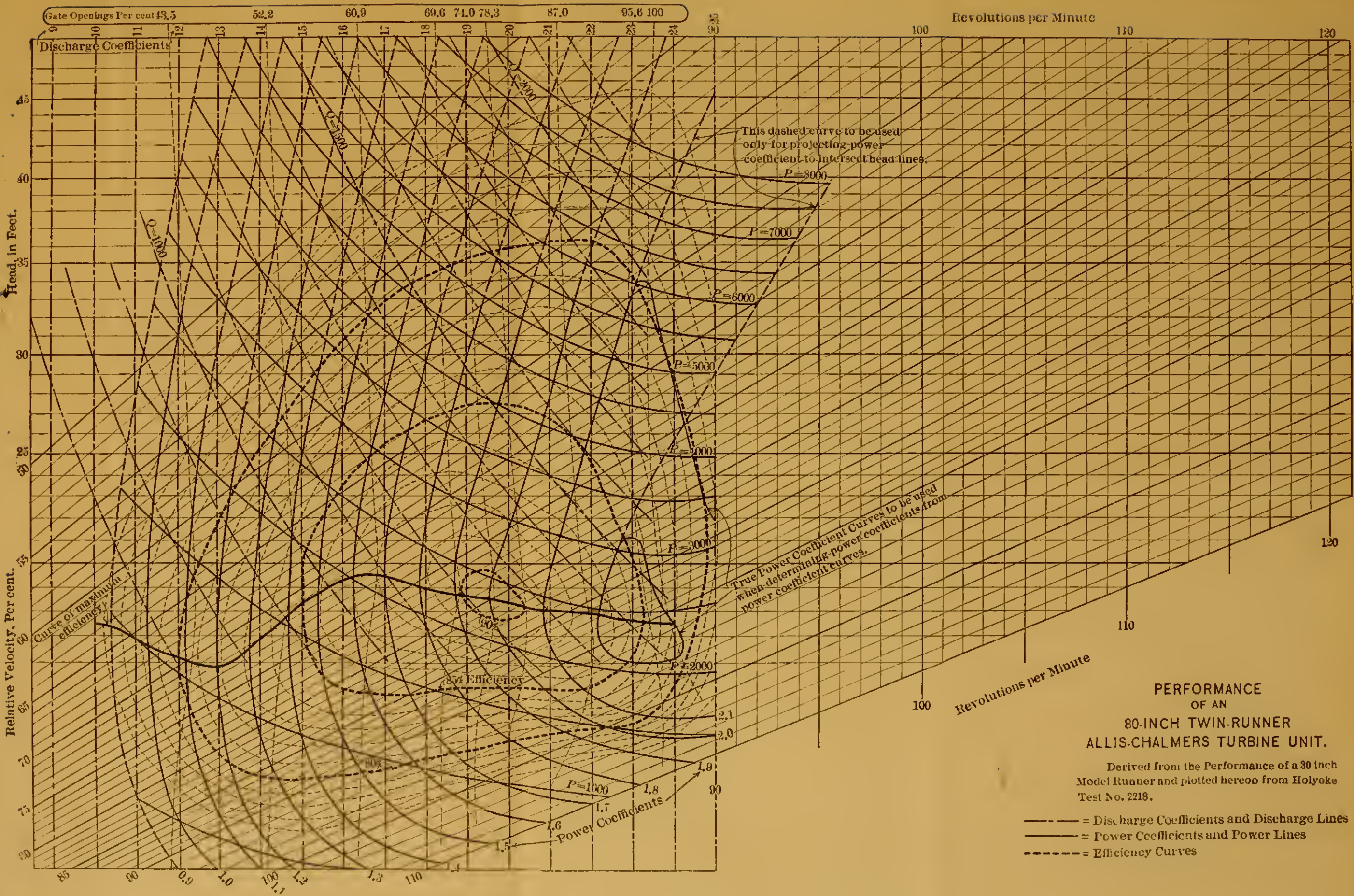
The gate opening, according to the Holyoke test, can be determined from the gate-opening curves, dotted on the diagram, by making the reading at the third point located on the diagram, at the intersection of the relative speed and power coefficient lines, in this case within the closed curve of 90% efficiency, as has been explained. This is about 72% of full gate, according to the plotting. Gate openings are of little use for purposes of this kind, but discharges are very important. Hence it is preferable to have the discharge coefficients

plotted as parallel vertical lines, rather than the gate openings, as it is necessary to project the locations of points in the direction of the discharge coefficient lines very frequently, and these projections can be most easily and correctly made with reference to parallel straight lines.

In the supervision of power-house operation, it is recommended that frequent, daily, or even hourly, reports of the operating conditions be made. These reports should give the output of each unit, with its maximum possible output under the conditions. The actual total output of the power plant, with its maximum possible output under the conditions, should also appear. Where these actual and maximum possible outputs show unreasonable discrepancies, an investigation should be started at once to ascertain the cause.

114.—Proper Speed and Load for Turbines.—In the case of alternating-current machinery, the units are usually restricted to a certain definite speed. In this case the greatest of care should be exercised in selecting turbines which will drive the generators at that speed, simultaneously operating at their maximum efficiency and best rated load. If the intending purchaser will have the makers guarantee performance according to a diagrammatic plotting such as outlined, perhaps limiting the guaranty to a certain operating area, or areas, of the diagram, he can then assure himself in advance of the efficiency which he can obtain at his proposed plant. A contract which simply requires a certain maximum efficiency, may fall far short of securing the desired results, for the reason that conditions may seldom admit of operation at that particular efficiency. The important relation of speed to power may be drawn off such a diagram very simply by first observing what relative speed line, or lines, pass through the most desirable field of conditions on the diagram. If the turbine can operate at its best load most of the time, the more restricted, smaller field of higher efficiencies can be made available; but, if the load must vary over a considerable range, it may be found that the higher efficiencies would seldom or never be available.

As an example of the relations between speed and load, Table 63 has been compiled from Plate LXVI, from which it appears that the turbine unit can develop 90% efficiency only between heads of 27.5 and 32.2 ft., if the speed must be held at 100 rev. per min. The best head is 30 ft., giving the widest range of power.



PERFORMANCE
OF AN
80-INCH TWIN-RUNNER ALLIS-CHALMERS TURBINE UNIT.

Derived from the Performance of a 30 Inch Model Runner and plotted hereon from Holyoke Test No. 2218.

——— = Discharge Coefficients and Discharge Lines
——— = Power Coefficients and Power Lines
------ = Efficiency Curves

TABLE 63.—RANGE OF POWER AT VARIOUS HEADS DEVELOPABLE BY ONE TURBINE UNIT AT 100 REV. PER MIN. AND 90% EFFICIENCY, AS SHOWN BY PLATE LXVI.

Head, in feet.	Horse-Power.		Range of horse-power.
	Least.	Highest.	
27.5	4 300	4 300	0
28.0	4 400	4 500	100
29.0	4 470	4 660	190
30.0	4 600	4 900	300
31.0	4 820	5 050	230
32.0	5 100	5 200	100
32.2	5 200 +	5 200 +	0

Again, when wider ranges of power are required with this unit, it must be at the expense of efficiency. Suppose it is desired to ascertain the range of power at 88% efficiency. Such an investigation appears in Table 64, which shows that a small sacrifice in efficiency secures a very much wider range of operating conditions, which will be found very necessary in many cases. Contracts, therefore, should have due regard to such requirements of operating conditions, and diagrams similar to Plate LXVI will govern the specifications for turbine performance.

TABLE 64.—RANGE OF POWER FOR ONE UNIT AT 100 REV. PER MIN. AND 88% EFFICIENCY, AS SHOWN BY PLATE LXVI.

Head, in feet.	Horse-Power.		Range of horse-power.
	Least.	Highest.	
24.8	3 800*	3 800*	0
26.0	3 800	4 100*	300
27.0	3 900	4 300	400
28.0	4 000	4 600	600
29.0	4 100	4 800	700
30.0	4 200	5 000	800
31.0	4 400	5 200	800
32.0	4 550	5 400	850
33.0	4 750	5 600	850
34.0	4 900	5 800	900
35.0	5 100	5 950	850
36.0	5 300	6 150	850
37.0	5 500	6 300	800
38.0	5 800	6 300	500
39.0	6 200	6 200	0

*Somewhat doubtful.

In cases where the speed of the units may be varied within certain limits, as with direct-current generators, or isolated alternating machines, it may be desirable to adjust the speed to the head and power, so as to secure maximum efficiency. Table 65 is a study of this kind compiled from Plate LXVI.

TABLE 65.—Adjusting the Speed to Head and Power when Feasible.

(The first item under any given power is the speed in revolutions per minute and the second item is the resulting maximum efficiency as given by Plate LXVI.)

Head, in feet.	Horse-Power.											
	3 500		4 000		4 500		5 000		5 500		6 000	
	Rev. per min.	Per-centage.	Rev. per min.	Per-centage.	Rev. per min.	Per-centage.	Rev. per min.	Per-centage.	Rev. per min.	Per-centage.	Rev. per min.	Per-centage.
25	89	90	98	88*	......	....	......	....	......	....	......	....
26	88	89	96½	90	......	....	......	....	......	....	......	....
27	87	88+	93	90+	......	....	......	....	......	....	......	....
28	87	87	93	89+	101	89+	......	....	......	....	......	....
29	88	86	92	88+	98	90+	105	88*	......	....	......	....
30	89	85	91	87+	97	89+	104	89+	......	....	......	....
31	91	84+	91	87	96	89	102	90+	108	88*+	......	....
32	94	83+	92	86	95	88	100	90+	108	89+	......	....
33	96	82+	94	85	94	87+	99	89	105	90+	111*	88
34	96	82	96	84+	95	86+	98	88+	103	90	111	89+
35	96	81	98	83+	97	85+	98	88	103	89	108	90+
36	96	80	100	83	98	85	98	87+	102	88+	106	90
37	96	79+	101	82	100	84	99	86	101	88	106	89
38	96	79	101	81+	103	83+	101	85+	100	87	105	88+
39	95	78	101	80+	104	83	102	85	102	86	104	88
40	95	77+	100	80	105	82	105	84	103	85	104	87+

* Doubtful if this could be fully realized.

In compiling a table of this kind, a line of maximum efficiencies for given power coefficients drawn on the diagram facilitates the compilation greatly. Thus, from the intersection of the given power and head, project the interpolated power-coefficient line by mental estimation until it intersects the curve of maximum efficiency. Enter in the table the efficiency reading from the efficiency scale. From the intersection last located and fixed, project the relative speed line by mental estimation until it intersects the given head line. Enter in the table the revolutions per minute, read from the speed scale, and proceed similarly with the next given head and power.

A similar table, taking into account generator losses and efficiencies, and thus relating to the performance of an entire unit, may be placed in the hands of the operator, with instructions to adjust the speed to the head and power according to the indications of the table.

Of course, a diagram based on the principles used in constructing the curves of Plate LXVI may be made, but taking into account, in addition, the performance characteristics of the generator, thus resulting in direct relations between the electrical output of the unit and the other quantities involved in Plate LXVI. For many purposes, such a diagram will prove to be of great value.

115.—Relations of Tests in Place to Holyoke Tests.—It is intended to make diagrams similar to Plate LXVI for the performance of the entire units, basing the data on the results of the tests, and taking into account, as fully as possible, the performance of the generators along with the turbines. Such a study will consume more time and space than the scope of the present paper will admit. Table 66, however, gives an approximate comparison of Holyoke tests with place tests based on assumed generator efficiencies. Owing to the uncertainty as to generator performance, no tests having been made on them, the comparison is only of passing interest.

It will be observed that there are relatively large efficiency corrections for Tests 32 and 39, Unit No. 13, due to relatively large deviations of the speed from 100 rev. per min. during the tests. It is doubtful if the speed was so much in error during these tests. Hence the corresponding discrepancies on Plate LXIII of the corrected plottings of these tests, indicated by open circles, are, very likely, excessive.

Attention is called to this, as the fact suggests the desirability of having a complete count of the total number of revolutions during each test, along with frequent speed determinations for shorter intervals. Had this been done, it would be possible to locate the cause of the discrepancies in the two plottings.

TABLE 66.—RELATIONS OF HOLYOKE TESTS TO TESTS IN PLACE.
(30 ft. head and 100 rev. per min.)

Test No.	Horse-power. General efficiency = 95 per cent.	Speed, in revolutions per minute.	Efficiency by test. General efficiency = 95 per cent.	Holyoke efficiency condition of test.	Comparison test efficiency to Holyoke efficiency.	Holyoke efficiency condition of 100 revolutions per minute.	Correction for plotting efficiency.
UNIT NO. 11.							
112	4 310	102.6	85.8	87.4	— 1.6	88.1	+ 0.7
113	4 364	100.0	87.2	88.3	— 1.1	88.3	0
114	4 431	100.5	87.4	88.5	— 1.1	88.55	+ 0.05
119	4 446	103.8	86.6	87.7	— 1.1	88.6	+ 0.9
115	4 481	101.3	87.5	88.5	— 1.0	88.75	+ 0.25
116	4 529	100.3	87.7	89.0	— 1.3	89.0	0
121	4 572	101.4	86.8	88.9	— 2.1	89.4	+ 0.5
118	4 594	98.0	88.7	89.8	— 1.1	89.5	— 0.3
122	4 610	101.0	89.0	89.4	— 0.4	89.6	+ 0.2
117	4 668	100.7	87.6	89.9	— 2.3	90.0	+ 0.1
120	4 748	101.7	86.8	90.1	— 3.3	90.3	+ 0.2

UNIT NO. 12.

Test No.	Horse-power. General efficiency, = 94 per cent.	Speed, in revolutions per minute.	Efficiency by test. General efficiency = 94 per cent.	Holyoke efficiency condition of test.	Comparison test efficiency to Holyoke efficiency.	Holyoke efficiency condition of 100 revolutions per minute.	Correction for plotting efficiency.
202	4 358	101.4	87.08	88.1	— 1.02	88.4	+ 0.3
206	4 450	99.1	88.42	88.8	— 0.38	88.6	— 0.2
207	4 409	101.1	87.99	88.3	— 0.31	88.5	+ 0.2
212	4 415	100.8	87.61	88.4	— 0.79	88.6	+ 0.2
201	4 517	100.3	89.10	88.9	— 0.20	88.9	0
204	4 479	98.8	88.82	88.9	— 0.08	88.7	— 0.2
208	4 453	100.1	87.67	88.7	— 1.03	88.7	0
211	4 418	100.0	86.63	88.6	— 1.97	88.6	0
205	4 489	101.1	87.61	88.6	— 0.99	88.8	+ 0.2
209	4 495	98.4	86.22	89.1	— 2.88	88.8	— 0.3
210	4 526	100.3	88.10	89.0	— 0.80	89.0	0
203	4 529	98.5	88.50	89.3	— 0.80	89.0	— 0.3
200	4 614	99.9	87.43	89.7	— 2.27	89.7	0
UNIT NO. 13.							
46	4 335	99.9	88.8+	88.2	+ 0.6	88.2	0
23	4 422	100.0	89.05	88.6	+ 0.45	88.6	0
33	4 425	99.1	89.0	88.7	+ 0.2	88.6	— 0.1
44	4 476	99.8	89.05	88.75	+ 0.30	88.75	0
41	4 496	99.6	89 12	88.9	+ 0.22	88.85	— 0.05
26	4 513	100.7	89.0	88.85	+ 0.15	88.9	+ 0.05
42	4 520	100.0	88.85	89.0	— 0.15	89.0	0
22	4 550	100.0	89.1	89.1	0.	89.1	0
32	4 580	98.7	88.8	89.6	— 0.8	89.3	— 0.3
39	4 586	101.0	88.8	89.1	— 0.3	89.4	+ 0.3
45	4 590	99.4	88.8	89.5	— 0.7	89.45	— 0.05
43	4 612	100.0	88.6	89.9	— 1.3	89.8	0
40	4 705	100.0	88.58	90.2	— 1.62	90.2	0

PART V.

CURRENT METERS.

SUMMARY.

116.—Current meters were run in parallel with the chemical equipment in a number of the earlier turbine tests as a precaution. These are marked: *D, E, G, H, I, K, L, N, O, S, T, U,* and *V.* Tests which were made with meters only are marked: *B, C, F, J, P, Q, R,* 1, 2, 3, 4, 5, 27, 28, 29, 30, 31, 34, 35, 36, 37, and 38. More than 4 000 individual velocity observations were made with the five current meters.

Ratings were made for these meters at the Naval Tank of the University of Michigan with the efficient equipment and expert advice of Professor H. C. Sadler. These ratings were made in still water, with and without oscillations. Oscillations of three kinds were used: longitudinal, vertical transverse, and horizontal transverse, of various degrees of amplitude and period. Figs. 32, 34, and 36 are examples.

The particular meters rated were: Haskell No. 1, Haskell No. 2, Ott No. 2311, Ott No. 2312, and Price No. 1553.

The following propositions, which are restatements of those previously made by the writer* are verified, with some modifications in the case of the small Price meter, under certain circumstances.

When a cup meter is run in turbulent water it will register a larger number of revolutions per second than a perfect still-water rating would indicate.

When a screw meter is run in turbulent water it will register a smaller number of revolutions per second than a perfect still-water rating would indicate.

If both types of meter are used simultaneously in turbulent water, the disparity between the discrepant velocities thus determined by the still-water rating may be taken as a basis for correcting the discrepant velocities.

In these ratings, the tails, or rudders, of the meters were removed, and the meters were rigidly attached to oscillating arms capable of motions in the three directions mentioned. Three meters were thus rated simultaneously, the three arms oscillating in unison and the meters pointing in the direction of motion of the car.

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* *Transactions,* Am. Soc. C. E., Vol. LXXVI, p. 819.

This method of holding the meters rigidly in one direction is very unfavorable to the screw meter and favorable to the cup meter, the result being that the screws are retarded in most cases by a larger amount, and the cups are accelerated by a smaller amount, than would be the case were the meters free to swing.

It appears to be the case that, for velocities greater than 1.5 ft. per sec., the relative errors of all types of meters are nearly, if not quite, constant for the same relative amount of oscillation or turbulence. This further appears to be nearly realized for velocities of less than 1.5 ft. per sec. Thus, in the case of Ott meter No. 2311, Fig. 34, the retardation is uniformly 5 to 6% for transverse oscillations averaging about 30% of the velocity of the rating car, with amplitudes of 2 or 3 ft.

In the head-races during the turbine tests two Haskell meters, two Ott meters, and one Price meter were attached to a rack, composed of angle irons, Fig. 25, Section 97, in exactly the same manner as they were attached to the rating car, the eddying motion of the water, however, taking the place of oscillations.

It was shown by Tests 24, 25, 70, 71, 72, and 73 that this rack had a retarding influence on the meters of a greater or less degree. It is clear that the rack would cut off the water flow in the obstructed part of the section, crowding it into the rest of the raceway. Thus a meter in front of the rack would record lower velocity and one in the unobstructed part of the section higher velocity than with the rack removed.

In the vertical transverse oscillating tests of the small Price meter, there are certain relative amounts of oscillation which have the effect of retarding the meter rather than accelerating it. Above and below this relative amount, acceleration results. To this extent the cups are retarded rather than accelerated, but, on the average, as in the usual gauging section, and as in the case of the present tests, it appears that the net effect on the meter was acceleration.

Longitudinal oscillations have very little effect on the still-water ratings of the meters. For this reason the longitudinal oscillations are neglected in the discharge computations.

117.—Current-Meter Tests.—It would not be rational to conclude this paper without making a comparison of the results by the chemical

method with those by current meters. It is of very great interest to learn how closely the two methods consist with each other. This comparison has a double value. It confirms the theories advanced for chemi-hydrometry and simultaneously shows how accurate current-meter work can be, under the very unfavorable conditions of flow in a head-race where no precautions have been taken for "stilling" the water for the purpose of gauging the discharge.

Although the two methods check each other well within 1%, it would scarcely be wise to assert that current meters, under these adverse conditions, will always yield results correct to a fraction of 1 per cent. It is clear, however, that, if meters of properly related types are used in conjunction with an intelligent statistical study of their performance, results of a remarkably high degree of precision can be obtained, even under very unfavorable hydraulic conditions.

No attempt will be made to effect a complete digest of all the current-meter observations. The object will not be to make the best possible use of the current-meter data, but rather to arrive at a hasty estimate of the discharge for each test where the current meters and chemicals were used in parallel, thus leading to an immediate comparison of the two methods.

118.—Assumptions.—It is always necessary to make assumptions when computing rates of discharge based on the records of current-meter observations. The usual method requires the assumption that the meters perform the same in turbulent water as in still water. This assumption is entirely too narrow for precise work, and has not only been controverted on many occasions by eminent engineers, but has been proved beyond question to leave the resulting estimates in doubt by a large margin.

The results will be the more reliable the more nearly the assumptions are in accord with the facts. In this series of tests, therefore, it is assumed that both horizontal transverse and vertical transverse oscillations of the meters in the ratings represent horizontal and vertical disturbances in the meter section during the actual discharge observations. The effects of longitudinal oscillations have been shown in the ratings to be very small. It is assumed as a consequence that the effects of longitudinal perturbations, or pulsations, in the meter section are also very small. Such effects, accordingly, are neglected.

The ratings at Ann Arbor show conclusively that, for the method of holding the meters, the small Price meter will, on the average, be accelerated, and the Haskell and Ott meters will be retarded, and that the Haskell meter will be retarded by a larger amount relatively than the Ott meter. These differences in the characteristics of these three types make determinate the problem of estimating the discharge with a considerable degree of precision. Two meters which perform exactly alike, thereby giving perfectly consistent results, however, are worthless for the purpose of measuring velocities in turbulent water with precision.

The fact, stated by W. G. Price, M. Am. Soc. C. E., in his discussion of the writer's former paper, that vertical disturbances (when the meter axis is vertical) tend to counteract the effects of horizontal components of motion, does not assist in determining true velocities by using two or more different types of meter, but, on the contrary, rather hinders the determination. This hindrance arises from the necessity of making diagrams for the purpose of separating the two classes of disturbances and, indeed, of determining, so to speak, the relative amounts of horizontal and vertical disturbance.

In this respect it would facilitate matters considerably to have meters each of which suffers equal deviations for equal degrees of either of these two classes of turbulence. The Haskell and Ott meters are of this kind, but both are retarded, whereas, for the most accurate work, the deviations should be in opposite directions. In the present case we must rely principally on the true velocities as determined by the two combinations, Haskell-Price and Ott-Price, though the combination, Haskell-Ott, must not be disregarded.

It may be repeated here that, by properly designing a runner of the screw type with the vanes slightly cupped in the right sense, it should be possible to construct a current meter giving records practically correct in all usual cases of turbulent flow. Mr. F. Nagler has made such an instrument, though his experimental study is not yet completed. The last runner produced gives records which are more nearly correct than those of any meter with which the writer is conversant.

119.—The Statistical Method.—The term "statistical" as here used relates to the law of averages as deduced from aggregates rather than from the study of each individual element in detail. The usual

method in discharge gauging is to find a number representing the velocity at a particular point in the meter section, and to take the product of this velocity and the area associated with the point as representative of the discharge through this area. The sum of all such products is the discharge for the entire cross-section. In the statistical method, the aggregate, or sum, of all the velocities in the meter section may be multiplied by the distribution factor* to determine at once the mean velocity in the section. The area of the section, as a function of the elevation of the water surface, taken from a table, or curve, may then be multiplied by the mean velocity to determine the total discharge, thus saving a large amount of detail.

The distribution factor is the ratio of the mean velocity in the section to the sum of the metered velocities at a fixed set of points in the cross-section. If V_m is the mean velocity in the section (unknown), A the aggregate meter velocity, that is, simply the sum of the metered velocities, N the number of meter points, and v_m the arithmetical mean of the meter velocities, the distribution factor is:

$$\text{Distribution factor} = \frac{V_m}{A} = \frac{V_m}{N\,v_m} \dots\dots\dots\dots (203)$$

Now, N is known, and the ratio, $V_m \div v_m$, is confined within very narrow limits for a large variety of cases if the meter points are distributed uniformly, or nearly uniformly, over the area. When the meter points are spaced uniformly across the usual river, the ratio is decreased to a somewhat larger extent, but even then it is pretty well confined to values between 0.92 and 0.96 for such sections as are chosen for gauging stations. For a liberal number of points distributed uniformly over the area of a large raceway, even when the conditions of flow are unfavorable, the ratio usually lies between 0.98 and 0.99, 0.985 being a close average. The tendency is for this value to increase with the size of the section. There may be cases, even in small sections, where it would be larger than unity. If the meters can be placed at points which determine the mean velocities throughout equal portions of the cross-section, the ratio of mean velocity to mean metered velocity becomes equal to unity, and the distribution factor is then simply the reciprocal of the number of velocities, that is, of meter points.

* Defined in the paper on current meters previously referred to. *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 837.

The value of the ratio $V_m \div v_m$, during the tests of 1911, was found, by a careful Pitot tube survey, to be 0.981 for the tail-races of the turbines.

120.—Subdivisions of Raceway.—The method of suspending and operating the current meters has been described fairly well in Part III. The equipment was arranged so that, by a simple adjustment, the meter points were placed at the centers of equal areas. The raceway was divided into 10 equal vertical and 10 equal horizontal strips, the 100 meter points being determined by the intersections of the 10 medians of the horizontal strips with the 10 medians of the vertical strips. A diagrammatic scheme is given herewith which represents the subdivisions of the raceway.

Diagrammatic Scheme for Meter Positions.

(Positions of meters on rack and positions of rack.)

Horizontals.	Depths.	1		2		3		4		5	
		W.	E.	W.	E.	W.	E.	W.	E.	W.	E.
1	1										
2	2 3										
3	4 5										
4	6 7										
5	8 9										
	10										

The rack carrying the five current meters could be lowered until the meters were at the level of any set of meter points. By shifting the movable leaf of the rack until it was brought into contact with the west wall of the race, the five meters took their respective westerly positions for the set of meter points. After the velocities were determined at these points, the rack was shifted to its easterly position, and the remaining five velocities for this level of the rack were determined. The rack was then lowered to the next level.

In order to reduce the labor of computation, the five positions and five horizontals, respectively located by consecutive pairs of verticals and horizontals, will be used, instead of the individual meter points. Thus the square representing the area for the easterly and westerly situations of the rack for any position (Position 2, for example) in combination with any horizontal (Horizontal 3, for example) contains four meter points, and the sum of the corresponding velocities will be frequently treated in the sequel rather than the velocity at any one of the four points. This means a reduction in subdivisions from 100 to 25.

It will be observed that any one traverse of the raceway enables any one meter to determine the twenty velocities in the vertical which are determined by the position of the meter on the rack. This vertical, or position, may be treated as one of the five equal parts into which the race is divided, and the sum of the twenty corresponding velocities may be used as an aggregate, thus reducing the number of equal subdivisions to 5 instead of 25.

The necessity for these subdivisions is brought about by the fact that the meters do not give true velocities. Otherwise the aggregate for the entire cross-section would be immediately available for the determination of discharge.

121.—Current-Meter Records.—Tests G, H, I, K, L, N, O, P, Q, R, S, T, U, and V.—Instead of giving the current-meter records in full, a somewhat condensed form will be adopted for the purpose of simplifying the calculations. There were 100 velocity readings in each of the tests excepting those in which two traverses of the head-race were made by the meters, in which case 200 velocity readings were made. Test *O* is the only one of the latter description. In this test the average of the two readings at any one point is treated exactly as were the single readings in the other tests.

Two readings are given by any meter at each depth, one for the easterly and one for the westerly position of the rack. The race is divided into five equal strips, each containing two depths at which the meters were read. These strips will be called "horizontals". Thus, for any one position and any one horizontal, there are four velocity readings, and it is the sum of these four readings which appears in the following assemblage of data under any position and opposite any horizontal. Decimal points have been omitted, except in the means for

each meter and position, at the foot of each column. For example, the sum of the four velocities in Test *G* by Haskell meter No. 1, in Position 2, Horizontal 3, is 1 030, or 10.3 ft. per sec. The sum of all the velocities in Position 2 by this meter is 4 949, or 49.49 ft. per sec., and the mean velocity is 2.474 ft. per sec.

The writer is indebted to H. W. King, M. Am. Soc. C. E., for the use of a small Price meter and to Mr. W. M. White for the use of two Ott meters. These, with the two large Haskell instruments, were used in the Ann Arbor ratings, and in these tests.

Tables of Tests *G, H, I, K, L, N, O, P, Q, R, S, T, U* and *V*, are given showing the sums of the four meter velocities for each of the twenty-five equal rectangles formed by the lapping of the five vertical rectangles representing the meter positions, with the five horizontal rectangles representing the five pairs of consecutive depths at which the meters were operated.

The numbers in the tables are in units of $\frac{1}{100}$ ft. per sec., decimal points having been omitted. The means are in feet per second.

Test *G*.—July 22d, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	O_1	H_1	P	H_2	O_2
1	1 098	1 033	1 038	1 015	1 068
2	1 227	1 051	1 058	959	1 020
3	1 238	1 030	1 033	986	1 034
4	1 071	1 011	992	982	1 186
5	828	824	946	809	720
Totals	5 462	4 949	5 067	4 751	5 028
Means	2.731	2.474	2.534	2.376	2.514

122.—Rating Current Meters.—In order to secure accurate results by current meters in turbulent water, it is necessary to use more than one type of meter, in the absence of an instrument which will not deviate from its still-water rating. Accordingly, three types were used in the tests: two of the Ott, two of the large-sized Haskell, and one of the small Price type. The letters, H, P, and O, will indicate the

corresponding types, and subscripts, where necessary, will indicate the particular number of the instrument referred to, according to the following scheme:

H_1 = Haskell meter No. 1,
H_2 = Haskell meter No. 2,
O_1 = Ott meter No. 2111,
O_2 = Ott meter No. 2112,
P = Price meter No. 1553.

TEST *II*.—JULY 24TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	O_2	O_1	H_1	P	H_2
1	1 162	1 032	980	1 105	1 088
2	1 166	1 041	1 041	1 114	1 315
3	1 120	1 011	1 060	1 137	1 173
4	1 076	1 000	881	1 161	587
5	886	855	699	1 134	512
Totals	5 410	4 939	4 661	5 651	4 675
Means	2.705	2.470	2.330	2.826	2.338

TEST *I*.—JULY 24TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	H_2	O_2	O_1	H_1	P
1	1 058	987	1 000	1 061	1 368
2	1 221	1 051	1 001	1 062	1 252
3	1 118	997	1 024	1 083	1 318
4	1 061	1 006	951	941	975
5	852	784	755	761	852
Totals	5 310	4 825	4 731	4 908	5 765
Means	2.655	2.412	2.366	2.454	2.882

TEST *K*.—JULY 24TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	P	H_2	O_2	O_1	H_1
1	1 149	1 038	1 037	1 094	1 009
2	1 242	1 037	1 046	1 058	1 014
3	1 268	1 011	1 004	1 029	1 128
4	1 056	988	926	1 048	736
5	936	794	706	816	465
Totals	5 651	4 868	4 719	5 045	4 352
Means	2.826	2.434	2.360	2.522	2.176

TEST *L*.—JULY 24TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	H_1	P	H_2	O_2	O_1
1	1 117	1 105	984	1 017	1 030
2	1 120	1 152	1 025	1 060	1 086
3	1 219	1 077	1 004	988	1 167
4	1 111	961	925	989	1 121
5	793	912	707	833	683
Totals	5 360	5 207	4 645	4 887	5 087
Means	2.680	2.604	2.322	2.444	2.544

Three meters could be rated simultaneously, with or without oscillations. Each instrument was attached to an arm at the rear of the rating car, and all three arms were tied together in such a manner that any oscillation imparted to any one meter would cause all three to oscillate in unison over equal amplitudes and in the same direction. In this way the separate effects of equal oscillations on all three meters could be recorded.

TEST *N*.—JULY 28TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	O_1	H_1	P	O_2	H_2
1	1 358	1 135	1 185	1 102	1 073
2	1 203	1 125	1 220	1 126	1 112
3	1 301	1 106	1 245	1 129	1 066
4	1 104	1 120	1 154	1 072	1 065
5	995	878	1 013	912	841
Totals	5 961	5 364	5 817	5 341	5 157
Means	2.980	2.682	2.908	2.670	2.578

TEST *O*.—JULY 30TH, 1914.

(Two traverses were made of the meter section in this test. The numbers are averages for the two traverses.)

Horizontal.	Position and Meter.				
	1	2	3	4	5
	H_2	O_1	H_1	P	O_2
1	1 255	1 186	1 116	1 160	1 142
2	1 165	1 198	1 134	1 192	1 121
3	1 275	1 163	1 097	1 194	1 091
4	1 220	1 066	1 080	1 144	1 053
5	925	963	842	1 104	840
Totals	5 840	5 576	5 269	5 794	5 247
Means	2.920	2.788	2.634	2.897	2.624

Oscillations of various amplitudes and periods were tried, and in three directions: longitudinal, vertical, and horizontal transverse. The axes of the meters were parallel to the direction of the motion of the car in all cases, except that, in the longitudinal oscillations, the arms naturally gave the axes of the meters slight rocking motions in vertical planes parallel to the motion of the car. The effects of these longitudinal oscillations have been neglected, as they were not large.

TEST *P*.—JULY 30TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	O_2	H_2	O_1	H_1	P
1	1 311	1 126	1 102	1 083	1 224
2	1 085	1 124	1 098	1 114	1 232
3	1 323	1 085	1 103	1 149	1 280
4	1 145	975	1 137	1 104	1 223
5	903	829	881	781	1 094
Totals	5 767	5 139	5 321	5 231	6 053
Means	2.884	2.570	2.660	2.616	3.026

TEST *Q*.—JULY 30TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	P	O_2	H_2	O_1	H_1
1	1 518	1 175	1 123	1 099	1 161
2	1 243	1 188	1 179	1 099	1 101
3	1 361	1 143	1 059	1 117	1 092
4	1 216	1 089	1 019	1 107	1 067
5	1 001	926	772	1 010	847
Totals	6 339	5 521	5 152	5 432	5 268
Means	3.170	2.760	2.576	2.716	2.634

As a general rule, the meters were retarded by longitudinal oscillations for such velocities as we now have to measure, so that no surprise will be warranted if the final estimates of discharge prove to be somewhat deficient, as compared with results by more accurate methods than have yet been devised for current meters.

Figs. 32, 33, 34, 35, and 36 give, each on one sheet, the complete series of ratings for one or other of the five meters. The method of constructing the curves, being simple, requires no explanation.

TEST *R*.—JULY 30TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	H_1	P	O_2	H_2	O_1
1	1 261	1 242	1 136	1 054	1 142
2	1 297	1 281	1 125	1 030	1 156
3	1 317	1 287	1 128	1 023	1 105
4	1 093	1 149	1 106	1 011	1 169
5	934	1 118	875	848	734
Totals	5 902	6 077	5 370	4 966	5 306
Means	2.951	3.038	2.685	2.483	2.653

TEST *S*.—AUGUST 8TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	O_1	H_1	P	O_2	H_2
1	1 208	1 178	1 079	1 002	1 024
2	1 290	1 088	1 113	1 047	944
3	1 310	1 069	1 117	1 044	898
4	1 068	1 047	1 078	1 006	1 003
5	909	875	972	778	637
Totals	5 785	5 257	5 359	4 877	4 506
Means	2.892	2.628	2.680	2.438	2.253

123.—Composite Rating Curves.—Plates LXVII, LXVIII, LXIX, LXX, and LXXI, are diagrams for determining the true velocity at a point by two velocity readings taken at this point by two meters of different type.

The true velocity may be read from the curved scale by locating a point on the diagram opposite the velocity, or revolutions per second, by one meter, and over the velocity, or revolutions per second, by the other meter, based on still-water ratings, thus representing the co-ordinates of the point on the diagram. The construction of the

TEST *T*.—AUGUST 10TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	H_2	O_1	H_1	P	O_2
1	1 023	917	936	1 065	1 131
2	1 202	1 038	987	1 067	1 022
3	1 169	1 035	978	1 060	1 066
4	1 076	1 050	929	1 040	1 097
5	871	921	727	1 013	646
Totals	5 341	4 961	4 557	5 245	4 962
Means	2.670	2.480	2.278	2.622	2.481

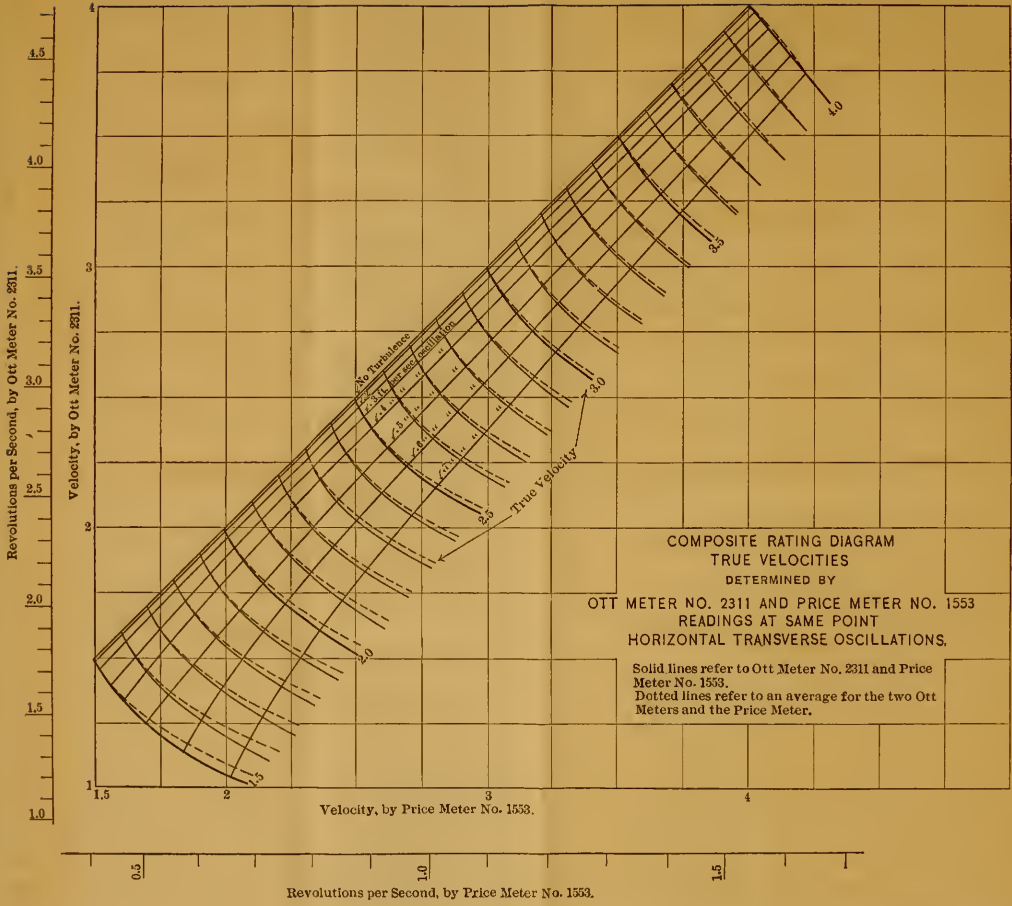
TEST *U*.—AUGUST 10TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	O_2	H_2	O_1	H_1	P
1	1 135	990	987	915	1 036
2	1 155	1 001	992	872	973
3	1 161	981	979	868	1 030
4	1 082	946	927	837	1 044
5	854	698	735	653	982
Totals	5 387	4 616	4 620	4 145	5 065
Means	2.694	2.308	2.310	2.072	2.532

diagrams from the ratings on Figs. 32, 34, and 36, will be sufficiently clear to the reader in principle, though, for accurate work, subsidiary diagrams segregating the ratings of different classes were made.

The curves on these subsidiary diagrams, when the ratings are not very complete, are not consistently spaced in all cases, and some minor adjustments were necessary to make them more harmonious and probably more nearly correct than they would have been otherwise.

It was never intended by the writer to make use of two meters which are each retarded, or each accelerated, to determine velocities in


COMPOSITE RATING DIAGRAM
TRUE VELOCITIES
DETERMINED BY
OTT METER NO. 2311 AND PRICE METER NO. 1553
READINGS AT SAME POINT
HORIZONTAL TRANSVERSE OSCILLATIONS.
Solid lines refer to Ott Meter No. 2311 and Price Meter No. 1553.
Dotted lines refer to an average for the two Ott Meters and the Price Meter.
No Turbulence
.3 ft. per sec. oscillation
True Velocity
Velocity, by Price Meter No. 1553.
Velocity, by Ott Meter No. 2311.
Revolutions per Second, by Ott Meter No. 2311.
Revolutions per Second, by Price Meter No. 1553.

TEST *V*.—AUGUST 12TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	P	O_2	H_2	O_1	H_1
1	1 232	1 037	993	986	983
2	1 229	1 012	954	950	931
3	1 064	1 033	971	1 045	863
4	1 132	966	879	1 024	705
5	982	798	669	842	508
Totals	5 639	4 846	4 466	4 847	3 990
Means	2.820	2.423	2.233	2.424	1.995

turbulent water. It was thought that, to secure the most accurate results, one meter should be accelerated and the other retarded. However, as the Haskell and Ott meters were both used in each test, a diagram of their composite rating, Plate LXXI, has been prepared. This diagram possesses the seeming inconsistency that the higher of two readings by the Haskell meter for a given reading by the Ott meter corresponds to the lower of the two velocities indicated by the diagram, and *vice versa*.

On each of the five diagrams of composite ratings the solid lines correspond to the composite ratings of only one pair of meters, and the broken lines correspond to the average composite ratings for the two types of meter represented, as determined from the Ann Arbor ratings.

Plates LXVII and LXIX are for horizontal transverse oscillations only, and Plates LXVIII and LXX are made from curves drawn midway between those for equal degrees of horizontal and vertical oscillations. The latter two diagrams, consequently, assume equal degrees of horizontal and vertical turbulence, but omit longitudinal turbulence, as has been explained in Section 122. The former pair of drawings assume only horizontal turbulence, omitting longitudinal and vertical perturbations. Plate LXXI is for the Haskell-Ott combination. As these meters are practically constant in form for all planes through their axes, they should be equally affected by equal

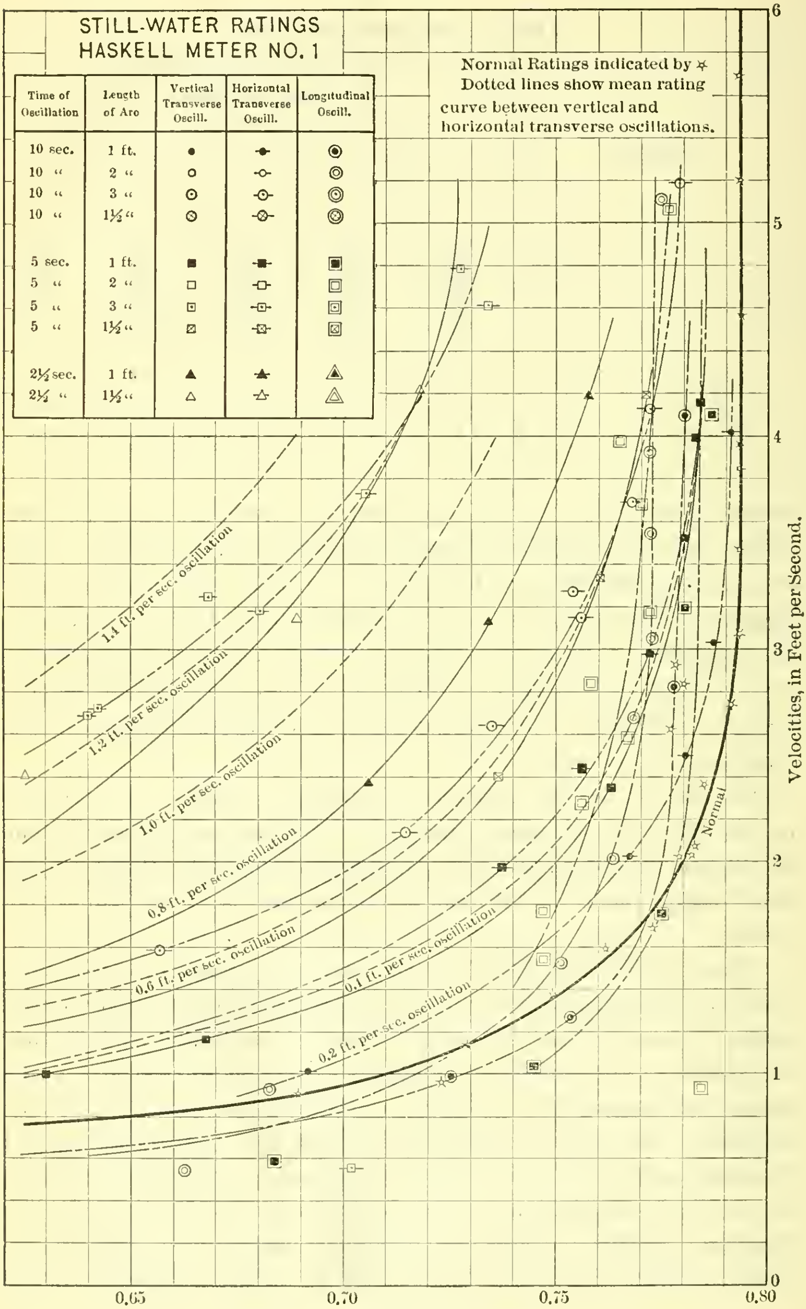
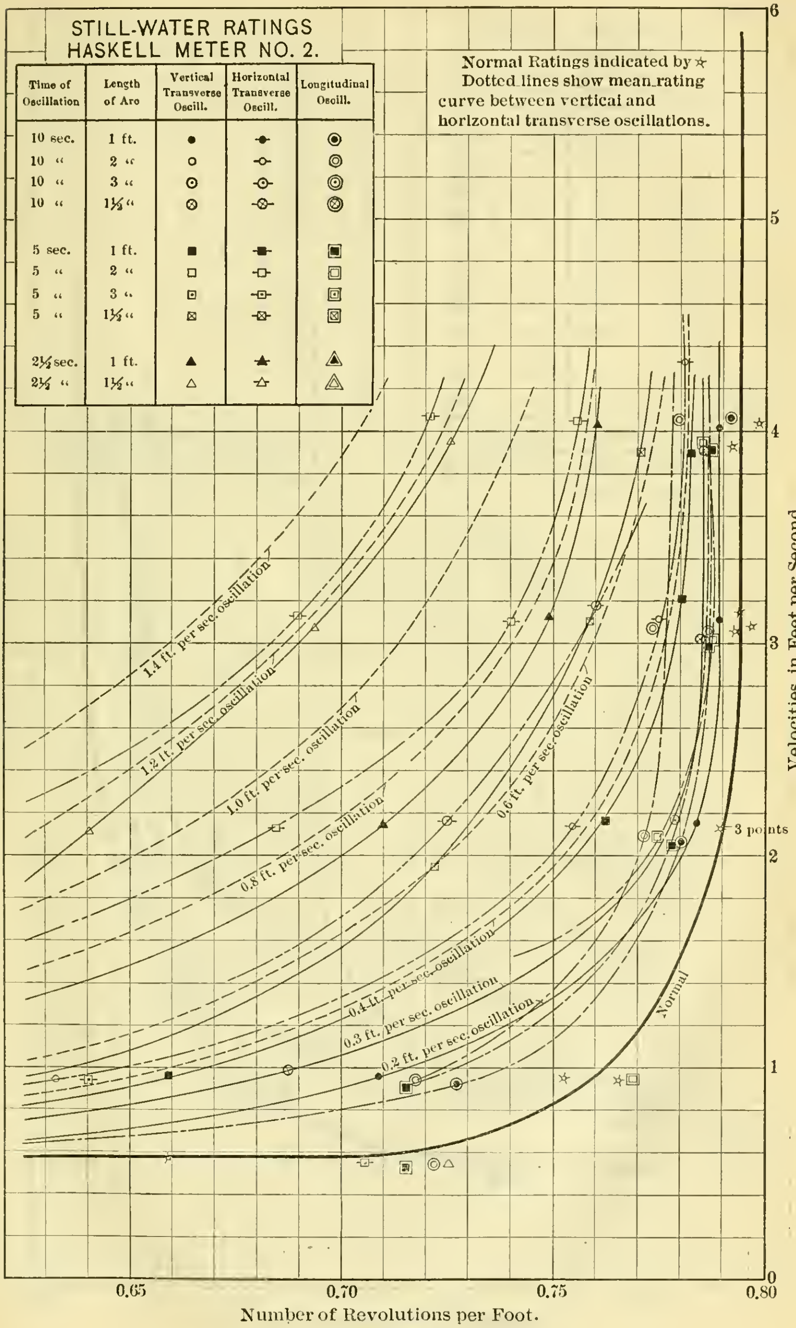

STILL-WATER RATINGS
HASKELL METER NO. 1
Time of Oscillation
Length of Arc
Vertical Transverse Oscill.
Horizontal Transverse Oscill.
Longitudinal Oscill.
10 sec. 1 ft.
10 " 2 "
10 " 3 "
10 " 1½ "
5 sec. 1 ft.
5 " 2 "
5 " 3 "
5 " 1½ "
2½ sec. 1 ft.
2½ " 1½ "
Normal Ratings indicated by ✡
Dotted lines show mean rating curve between vertical and horizontal transverse oscillations.
1.4 ft. per sec. oscillation
1.2 ft. per sec. oscillation
1.0 ft. per sec. oscillation
0.8 ft. per sec. oscillation
0.6 ft. per sec. oscillation
0.4 ft. per sec. oscillation
0.2 ft. per sec. oscillation
Normal
Velocities, in Feet per Second.
0
1
2
3
4
5
6
0.65
0.70
0.75
0.80
Number of Revolutions per Foot.

FIG. 32.



STILL-WATER RATINGS
HASKELL METER NO. 2.
Time of Oscillation
Length of Arc
Vertical Transverse Oscill.
Horizontal Transverse Oscill.
Longitudinal Oscill.
10 sec.
1 ft.
10 "
2 "
10 "
3 "
10 "
1½ "
5 sec.
1 ft.
5 "
2 "
5 "
3 "
5 "
1½ "
2½ sec.
1 ft.
2½ "
1½ "
Normal Ratings indicated by ☆
Dotted lines show mean rating curve between vertical and horizontal transverse oscillations.
1.4 ft. per sec. oscillation
1.2 ft. per sec. oscillation
1.0 ft. per sec. oscillation
0.8 ft. per sec. oscillation
0.6 ft. per sec. oscillation
0.4 ft. per sec. oscillation
0.3 ft. per sec. oscillation
0.2 ft. per sec. oscillation
Normal
3 points
Velocities, in Feet per Second
0
1
2
3
4
5
6
0.65
0.70
0.75
0.80
Number of Revolutions per Foot.

FIG. 33.

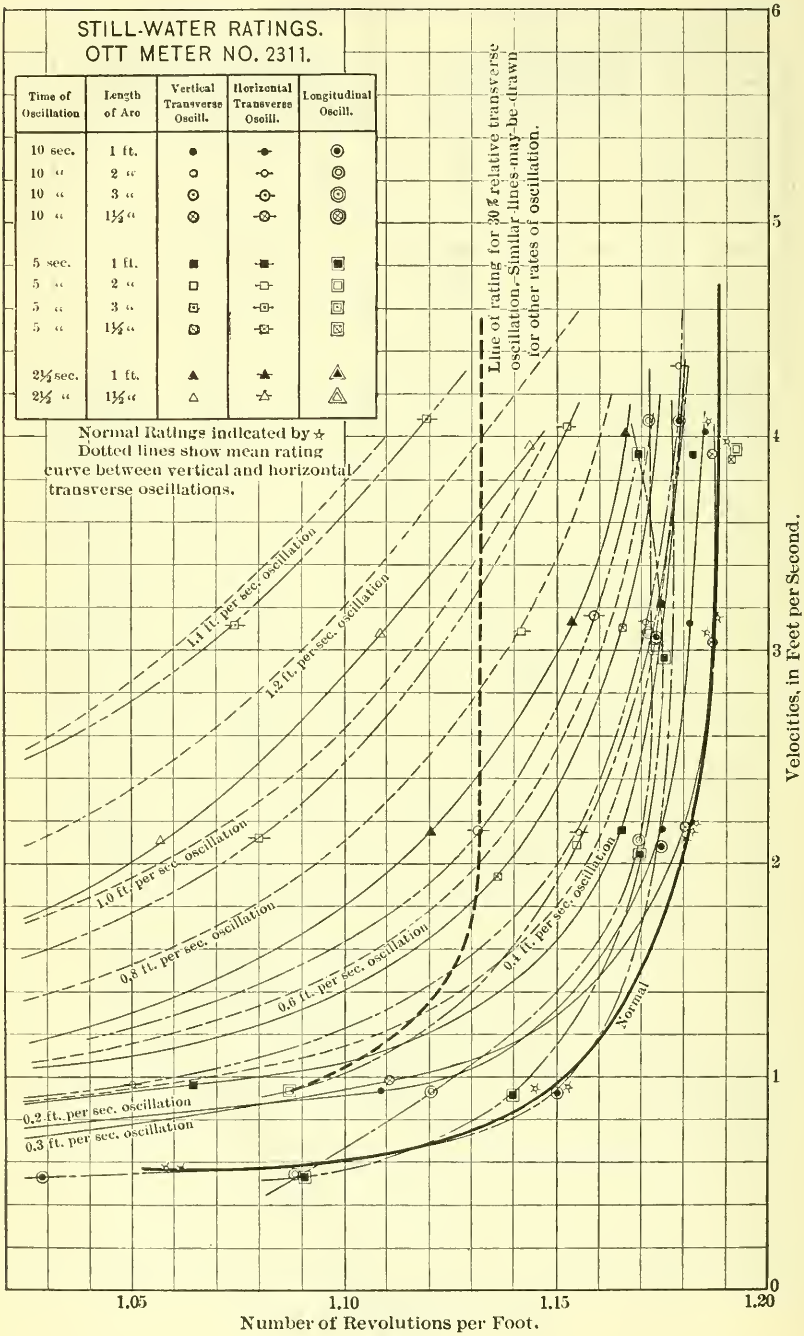

STILL-WATER RATINGS.
OTT METER NO. 2311.
Time of Oscillation
Length of Arc
Vertical Transverse Oscill.
Horizontal Transverse Oscill.
Longitudinal Oscill.
10 sec. 1 ft.
10 " 2 "
10 " 3 "
10 " 1½ "
5 sec. 1 ft.
5 " 2 "
5 " 3 "
5 " 1½ "
2½ sec. 1 ft.
2½ " 1½ "
Normal Ratings indicated by ☆
Dotted lines show mean rating curve between vertical and horizontal transverse oscillations.
Line of rating for 30 % relative transverse oscillation. Similar lines may be drawn for other rates of oscillation.
1.4 ft. per sec. oscillation
1.2 ft. per sec. oscillation
1.0 ft. per sec. oscillation
0.8 ft. per sec. oscillation
0.6 ft. per sec. oscillation
0.4 ft. per sec. oscillation
0.2 ft. per sec. oscillation
0.3 ft. per sec. oscillation
Normal
Velocities, in Feet per Second.
0
1
2
3
4
5
6
1.05
1.10
1.15
1.20
Number of Revolutions per Foot.

FIG. 34.

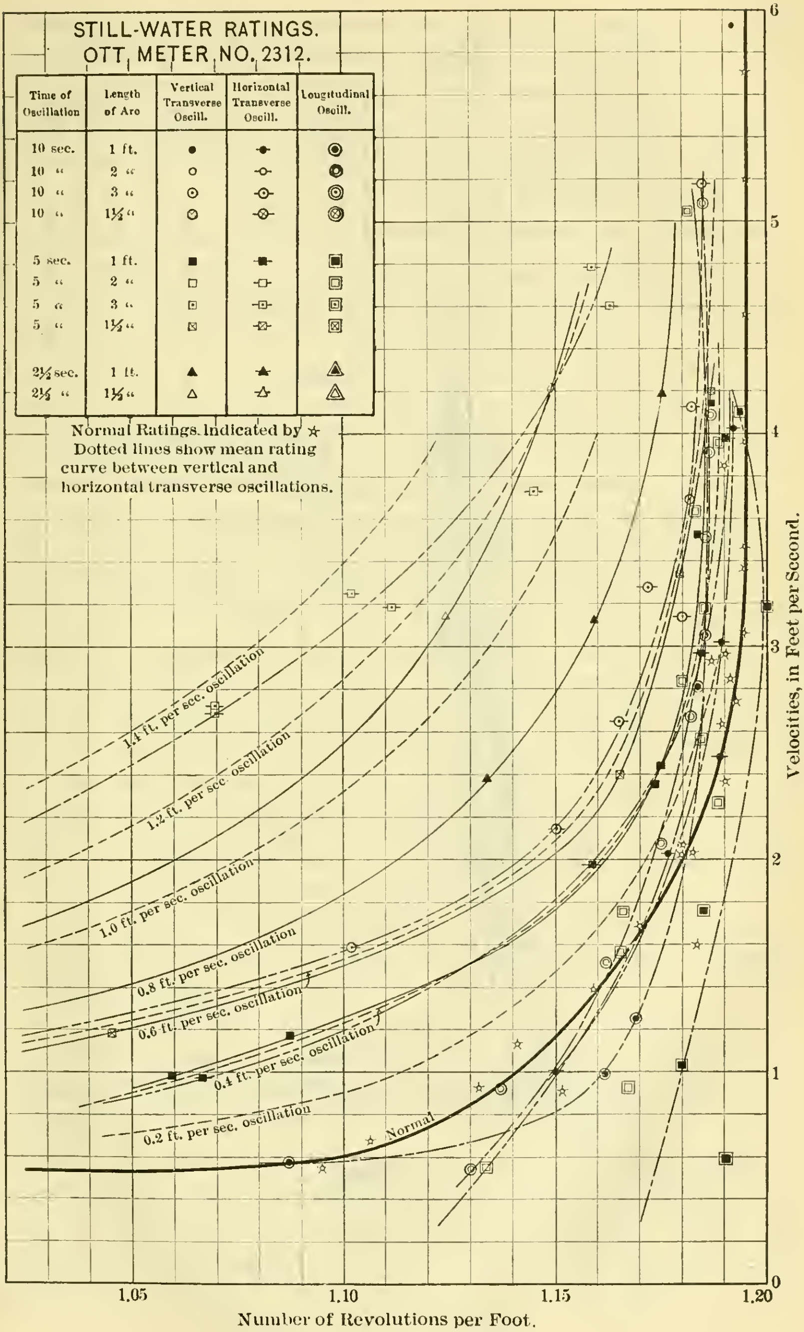

STILL-WATER RATINGS.
OTT METER NO. 2312.
Time of Oscillation
Length of Arc
Vertical Transverse Oscill.
Horizontal Transverse Oscill.
Longitudinal Oscill.
10 sec.
1 ft.
10 "
2 "
10 "
3 "
10 "
1½ "
5 sec.
1 ft.
5 "
2 "
5 "
3 "
5 "
1½ "
2½ sec.
1 ft.
2½ "
1½ "
Normal Ratings Indicated by ☆
Dotted lines show mean rating curve between vertical and horizontal transverse oscillations.
1.4 ft. per sec. oscillation
1.2 ft. per sec. oscillation
1.0 ft. per sec. oscillation
0.8 ft. per sec. oscillation
0.6 ft. per sec. oscillation
0.4 ft. per sec. oscillation
0.2 ft. per sec. oscillation
Normal
Velocities, in Feet per Second.
0
1
2
3
4
5
6
1.05
1.10
1.15
1.20
Number of Revolutions per Foot.

FIG. 35.

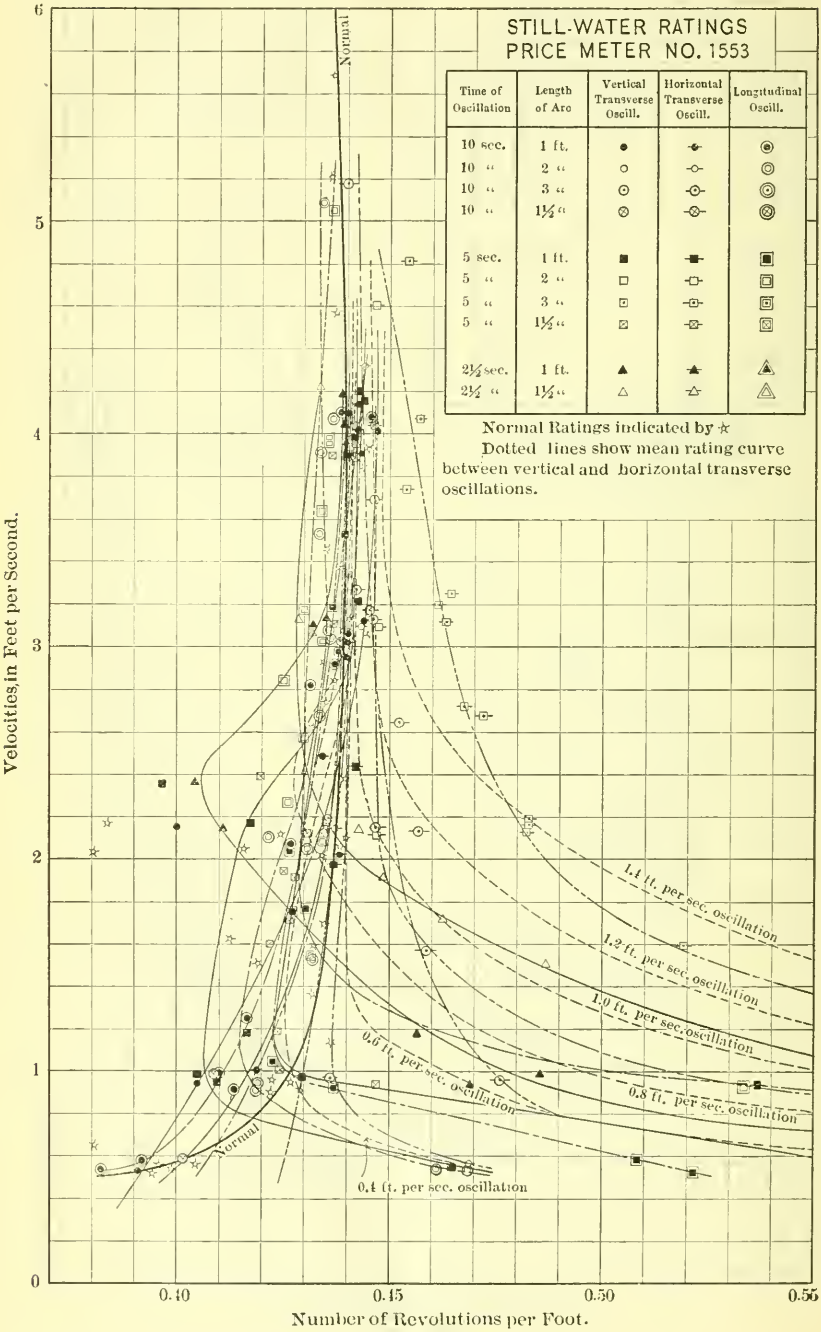

STILL-WATER RATINGS
PRICE METER NO. 1553
Time of Oscillation
Length of Arc
Vertical Transverse Oscill.
Horizontal Transverse Oscill.
Longitudinal Oscill.
10 sec. 1 ft.
10 " 2 "
10 " 3 "
10 " 1½ "
5 sec. 1 ft.
5 " 2 "
5 " 3 "
5 " 1½ "
2½ sec. 1 ft.
2½ " 1½ "
Normal Ratings indicated by ☆
Dotted lines show mean rating curve between vertical and horizontal transverse oscillations.
Normal
1.4 ft. per sec. oscillation
1.2 ft. per sec. oscillation
1.0 ft. per sec. oscillation
0.8 ft. per sec. oscillation
0.6 ft. per sec. oscillation
0.4 ft. per sec. oscillation
Velocities, in Feet per Second.
Number of Revolutions per Foot.
0
1
2
3
4
5
6
0.40
0.45
0.50
0.55

Fig. 36.

degrees of any kind of transverse turbulence, horizontal or vertical. The tests show this to be the case. Hence only one diagram is necessary for this combination of meters. It is the change in rating of the small Price meter in passing from horizontal to vertical turbulence which necessitates the two diagrams when this instrument is involved, though it must be remarked, in justice to this meter, that it is, on the average, for the method of suspension and operation used on these tests, relatively nearer to its still-water rating than any of the others.

124.—Velocities in Tests G, H, I, K, and L, Based on Meter Corrections Deduced from Average Meter Velocities in the Five Positions in These Tests.—These five tests were intended to determine the meter corrections. They were made under very uniform conditions, and are nearly identical, except that the meters were shifted in a systematic manner from test to test, so that each instrument made the traverse of a different position in each succeeding test, thereby operating once at the intersection of any vertical position with any horizontal, that is, at 100 different points of the head-race, in the five tests.

Instead of comparing each pair of meters at each point in the head-race, thus necessitating a large amount of detail, the aggregate velocity for each test has been computed from the mean velocities given by the partial aggregates for the separate meters in each position.

Table 67 gives the average reading for each meter in each test for the 20 meter points in the position in which the particular meter was operated, reduced to the common discharge of 1 660 cu. ft. per sec., and to the common section area of 625 sq. ft. See Section 121 for the data from which the table is compiled.

Table 68 is a rearrangement of Table 67 for the purpose of assembling the records for each meter in each position, regardless of the particular test involved.

Table 69 determines the true average velocities for the five meters in the five positions, regardless of the particular test. In compiling this table, Plates LXVII, LXVIII, LXIX, LXX, and LXXI, were used.

It is at once evident that the two velocities for any position determined for horizontal and vertical oscillations agree within about 1% of each other, as an average. The same may be said for the case of horizontal oscillations only.

TABLE 67.—EQUIVALENT MEAN INSTRUMENTAL VELOCITIES IN TESTS *G, H, I, K,* AND *L,* FOR A DISCHARGE OF 1 660 CU. FT. PER SEC. AND AN AREA OF 625 SQ. FT., BASED ON STILL-WATER RATINGS. UNIT NO. 13.

Position number.	Test.									
	G		*H*		*I*		*K*		*L*	
1	O_1	2.73	O_2	2.71	H_2	2.65	P	2.83	H_1	2.71
2	H_1	2.48	O_1	2.47	O_2	2.41	H_2	2.44	P	2.63
3	P	2.53	H_1	2.33	O_1	2.36	O_2	2.37	H_2	2.34
4	H_2	2.38	P	2.83	H_1	2.45	O_1	2.53	O_2	2.47
5	O_2	2.52	H_2	2.34	P	2.88	H_1	2.18	O_1	2.57
Section area	625.8		625.8		625.2		623.9		626.0	
Approx. discharge.	1 662		1 662		1 666		1 653		1 647	
Factor*	1.001		1.001		0.997		1.003		1.010	

* This factor was used to reduce the meter record tests *G H, I, K,* and *L,* Section 121, to values corresponding to a discharge of 1 660 cu. ft. per sec. and an area of 625 sq. ft. The corrections are so small that they are frequently omitted.

It may be said, therefore, that the two velocities determined by the Haskell-Price and Ott-Price combinations agree fairly well.

Velocities by the Haskell-Ott combination, however, are considerably deficient, as an inspection of the fifth line of velocities in Table 69 shows. This is undoubtedly due to some change in the relations between the two meters. The Ott meter appears to read lower than it should, according to the ratings, as all the velocities for the combination which include the Ott meter are low.

It was stated in Section 123 that it was not intended to use velocities determined by the Haskell-Ott combination, because the two screw meters are so much alike in performance that errors become magnified. Hence the velocities by this combination are untrustworthy, and should be neglected, except for purposes of comparison.

The problem then narrows itself to the determination of correct average velocities by the separate combinations of the Haskell and Ott meters with the Price.

In the case of the diagrams for horizontal oscillations, it is clear that, if we should wish to alter them by introducing vertical components, the errors of the Price meter would be reduced and those

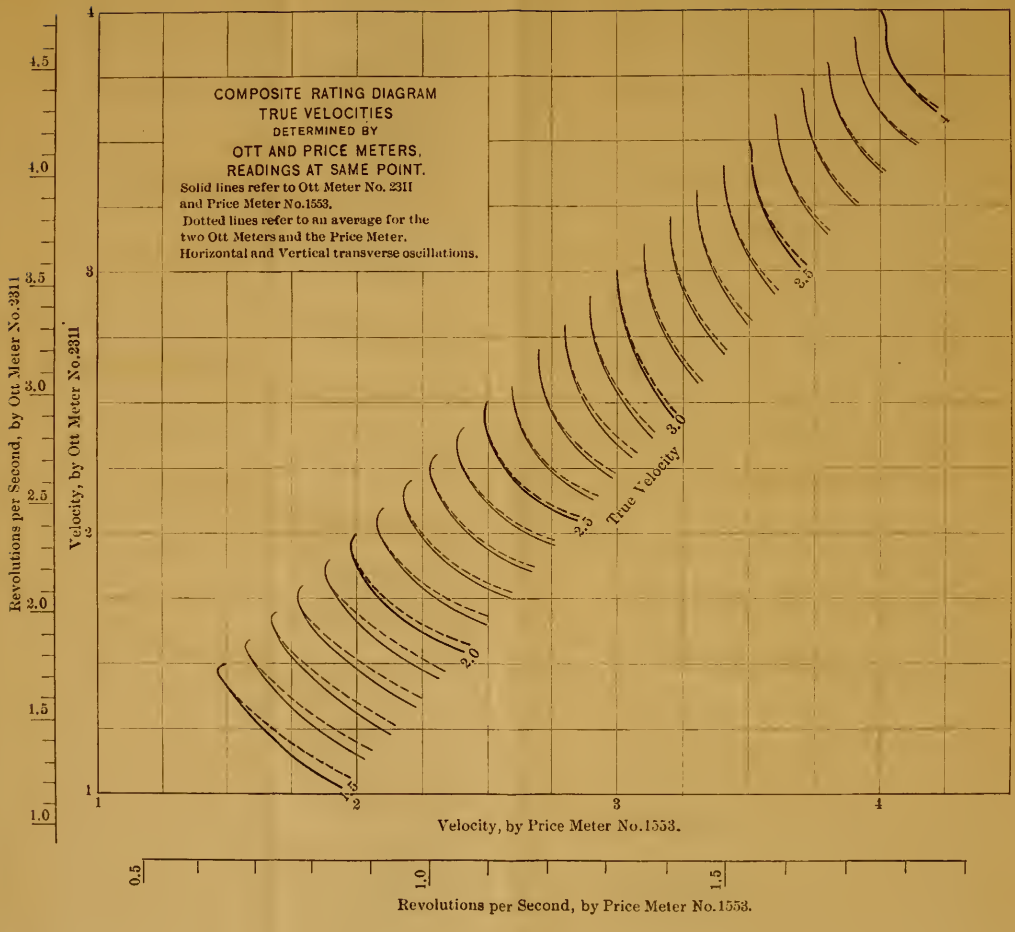

COMPOSITE RATING DIAGRAM
TRUE VELOCITIES
DETERMINED BY
OTT AND PRICE METERS,
READINGS AT SAME POINT.
Solid lines refer to Ott Meter No. 2311
and Price Meter No.1553.
Dotted lines refer to an average for the
two Ott Meters and the Price Meter.
Horizontal and Vertical transverse oscillations.
True Velocity
2.0
2.5
3.0
3.5
Velocity, by Ott Meter No.2311
Revolutions per Second, by Ott Meter No.2311
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
1
2
3
4
Velocity, by Price Meter No.1553.
0.5
1.0
1.5
Revolutions per Second, by Price Meter No.1553.

TABLE 68.—MEAN INSTRUMENTAL VELOCITIES OF THE FIVE METERS IN THE FIVE POSITIONS FOR TESTS *G, H, I, K*, AND *L*.—A REARRANGEMENT OF TABLE 67.

(Common discharge = 1 660 cu. ft. per sec. Common area = 625 sq. ft.)

Meter.	Position.				
	1	2	3	4	5
Price	2.83	2.63	2.53	2.83	2.88
$Haskell_1$	2.71	2.48	2.33	2.45	2.18?
$Haskell_2$	2.65	2.44	2.34	2.38	2.34
Ott_1	2.73	2.47	2.36	2.53	2.57
Ott_2	2.71	2.41	2.37	2.47	2.52
Average Haskell	2.68	2.46	2.335	2.415	2.26?
Average Ott	2.72	2.44	2.365	2.50	2.545

TABLE 69.—TRUE AVERAGE VELOCITY DETERMINED BY PLATES LXVII, LXVIII, LXIX, LXX, AND LXXI, FROM TABLE 68.

(Common discharge = 1 660 cu. ft. per sec. Common area = 625 sq. ft.)

Combination of meters.	Diagram.	Kind of oscillation.	Position.				
			1	2	3	4	5
$H-P_1$	Plate LXX	Horizontal and vertical.	2.83	2.64	2.55	2.81	2.80 (2.84)
H-P	" LXIX	Horizontal	2.80	2.61	2.50	2.73	2.70
O-P	" LXVIII.	Horizontal and vertical	2.83	2.61	2.52	2.76	2.81
O-P	" LXVII	Horizontal	2.79	2.56	2.47	2.70	2.74 (2.75)
O-H	" LXXI		2.76	2.45?*	2.40	2.59	3.00
Average of all five			2.802	2.574	2.488	2.718	(2.776) 2.81
Average of upper four			2.812	2.605	2.51	2.75	(2.782) 2.762

* Meter readings are discordant. Ott should exceed Haskell. Mean of meter velocities taken, and this must be too low.

Haskell meter seems to run slow in Position 5. Test *K*. The figure in parentheses represents a possibly more correct value.

of the screw meters would be numerically increased. This would cause the true velocity curve scale to rotate about the true velocity points for zero errors (on the 45° line) in clockwise direction. Hence, if actual vertical components are present in the head-race, velocities

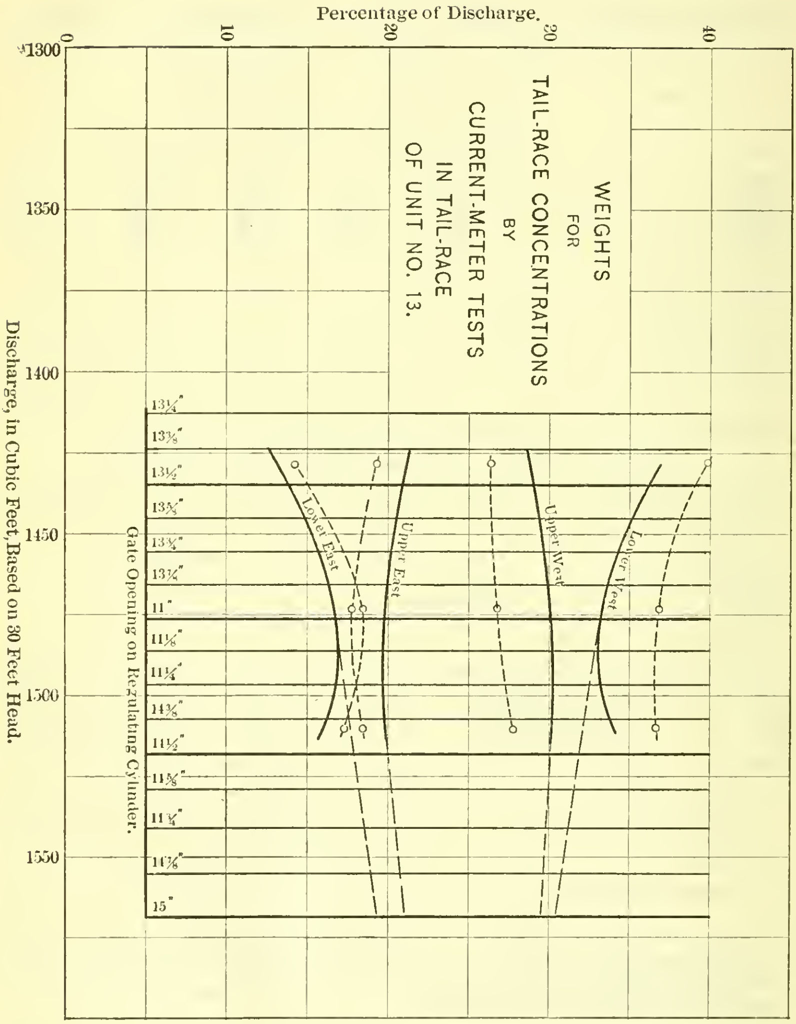
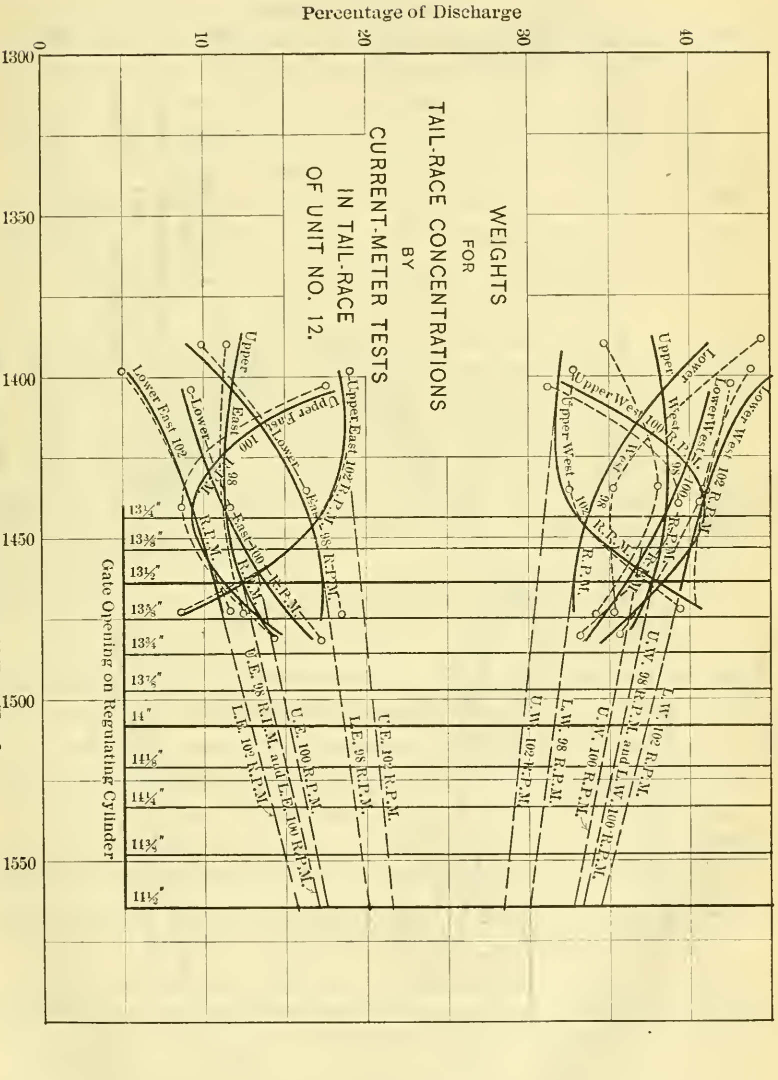

WEIGHTS
FOR
TAIL-RACE CONCENTRATIONS
BY
CURRENT-METER TESTS
IN TAIL-RACE
OF UNIT NO. 13.
Percentage of Discharge.
0
10
20
30
40
1300
1350
1400
1450
1500
1550
Discharge, in Cubic Feet, Based on 30 Feet Head.
Gate Opening on Regulating Cylinder.
Lower East
Upper East
Upper West
Lower West

FIG. 37.



Discharge, in Cubic Feet, Based on 30 Feet Head.

FIG. 38.

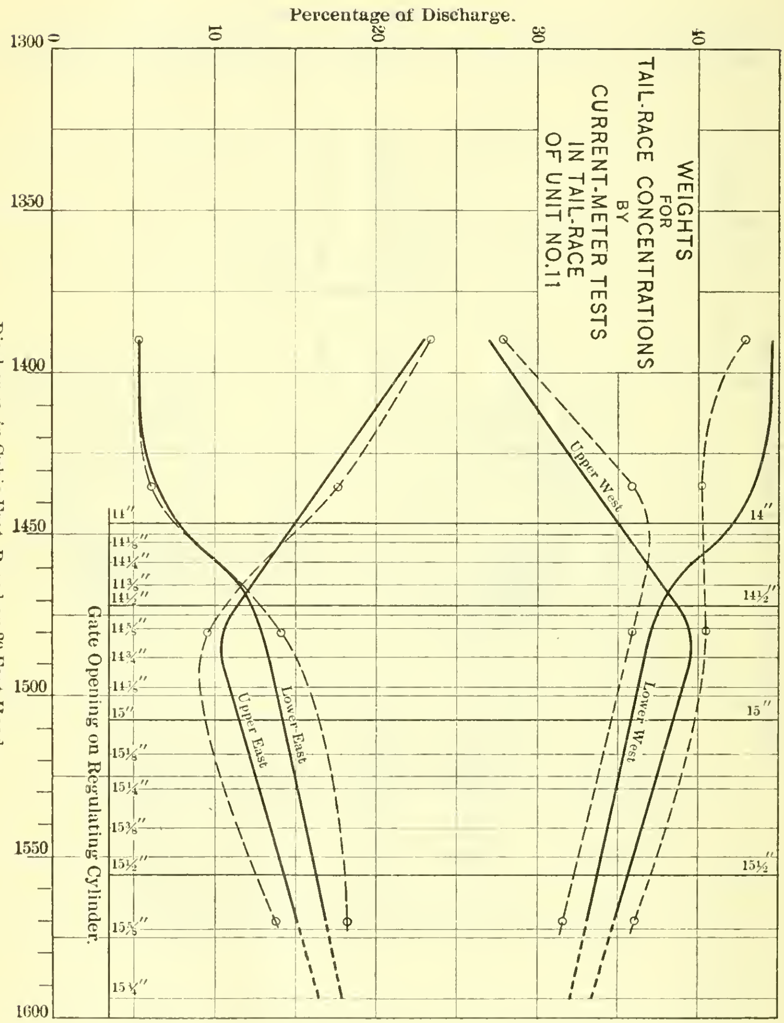

Percentage of Discharge.
WEIGHTS
FOR
TAIL-RACE CONCENTRATIONS
BY
CURRENT-METER TESTS
IN TAIL-RACE
OF UNIT NO.11
Upper West
Lower West
Upper East
Lower East
Gate Opening on Regulating Cylinder.
Discharge, in Cubic Feet, Based on 30 Feet Head.

FIG. 39.

determined by the diagrams for horizontal oscillations will be too low. There are undoubtedly vertical components of motion in the head-water. Therefore, all the velocities by the diagrams for horizontal disturbance should be too low, rather than too high.

By similar reasoning, it may be shown that, in the case of the diagrams for horizontal and vertical components, if too little oscillation has been assumed for the screw meters in constructing the diagrams, the curve scales will lie counter clockwise from their proper positions. In constructing these diagrams, only the horizontal components have been considered in plotting the co-ordinates for the Haskell and Ott meters. Therefore, velocities by these diagrams, also, will tend to be too low, supposing the correct values of the components have been used in plotting the co-ordinates for the Price meter during the construction of the diagrams.

Moreover, all readings of the meters are likely to be low, as longitudinal oscillations have been neglected altogether. It is not likely, therefore, that any of the velocities in Table 69 are too high.

On the other hand, if too much vertical component has been assumed in constructing the diagrams, the lines would lie clockwise from their correct positions, thus resulting in velocities which are too high.

Now, there was probably not as much vertical as horizontal disturbance in the water, so one element is present which would tend to make the readings by the diagrams for horizontal and vertical oscillations too high. On the whole, it might be assumed, without a serious probable error, that the velocities by the diagrams for horizontal and vertical components will give results which are nearest to the truth.

There will be no serious objection, however, to averaging the velocities in the first four lines of Table 69, as no great differences are involved. In this way a possible systematic error of some magnitude may be reduced. Accordingly, an average of those velocities for each position will be taken as the best probable value.

We are now in a position to compute the factors by which the velocities of the various meters for each position must be multiplied to secure true velocities. The true average velocities at the foot of Table 69 are to be divided by the average velocities of each type of meter from Table 68 to determine the factors. The result is given in Table 70.

TABLE 70.—CORRECTION FACTORS FOR THE METERS, BASED ON THE COMPUTED TRUE VELOCITIES FOR THE FIVE METER POSITIONS IN TESTS *G, H, I, K,* AND *L,* DETERMINED BY PLATES LXVII, LXVIII, LXIX, AND LXX.

(Common discharge = 1 600 cu. ft. per sec. Common area = 625 sq ft.)

Meter.	Position of Meter.					
	1	2	3	4	5	Mean.
Price	0.994	0.991	0.992	0.972	(0.967) 0.960	0.982
Haskell	1.049	1.059	1.075	1.139	(1.179) 1.222	1.109
Ott	1.034	1.068	1.061	1.100	(1.093) 1.085	1.070
Mean						1.054

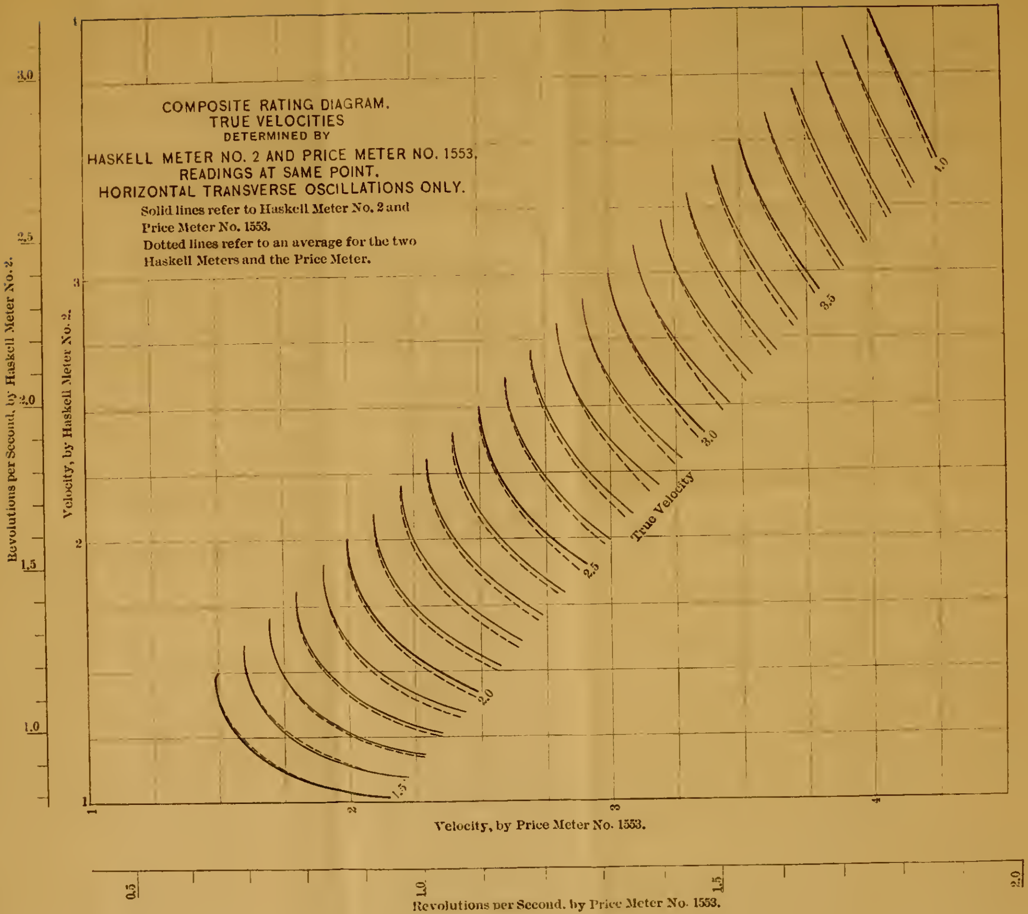
NOTE.—The parentheses refer to a possibly more correct value. See foot-note to Table 69.

TABLE 71.—SUMS OF THE TRUE VELOCITIES AT THE 100 METER POINTS IN TESTS *G, H, I, K,* AND *L,* BASED ON FACTORS DETERMINED FROM THE PERFORMANCE OF THE METERS IN THE FIVE POSITIONS DURING THESE TESTS.

Position.	Test.				
	G	*H*	*I*	*K*	*L*
1	565	559	557	562	562
2	524	528	515	516	516
3	503	501	502	501	499
4	541	549	559	555	538
5	(550) 546	(552) 572	(558) 553	(513) 532	(556) 552
Sum of velocities at the 100 meter points in head-race.	(268.3) 267.9	(268.9) 270.9	(269.1) 268.6	(264.7) 266.6	(267.1) 266.7

NOTE.—The figures in parentheses are based on a possibly more correct value for meter velocity. See foot-note to Table 69.

Turning now to the tests in Section 121, it is a simple matter to compute all the sums of the true velocities for the five positions of each test by multiplying the five totals by the corresponding factors from Table 70. This process leads to Table 71, the totals therein being the computed sums of the velocities at the 100 meter points,


COMPOSITE RATING DIAGRAM.
TRUE VELOCITIES
DETERMINED BY
HASKELL METER NO. 2 AND PRICE METER NO. 1553.
READINGS AT SAME POINT.
HORIZONTAL TRANSVERSE OSCILLATIONS ONLY.
Solid lines refer to Haskell Meter No. 2 and
Price Meter No. 1553.
Dotted lines refer to an average for the two
Haskell Meters and the Price Meter.
True Velocity
1.5
2.0
2.5
3.0
3.5
4.0
Velocity, by Price Meter No. 1553.
1
2
3
4
Revolutions per Second, by Price Meter No. 1553.
0.5
1.0
1.5
2.0
Velocity, by Haskell Meter No. 2.
Revolutions per Second, by Haskell Meter No. 2.
3.0
2.5
2.0
1.5
1.0

based on the factors determined in Table 70 from a comparison of velocities by the different meters in each position of Tests *G, H, I, K,* and *L.*

125.—Velocities in Tests G, H, I, K, and L, Based on Meter Corrections Deduced from Average Meter Velocities in the Five Horizontals in These Tests.—To show that we have a fairly stable method of computing factors for connecting the meter velocities, it will be advisable to compute them for the five horizontals, using the data from the same tests treated in the preceding section. It will not be necessary, however, in the case of horizontals, to reduce to a common discharge and common area, as each meter operates once in each position during the five tests. Variations due to differences in conditions among the tests tend to affect all meters in a similar sense, thus eliminating, on the average, any serious inconsistencies due to this cause. The average velocity for each meter in a given horizontal for all five tests is computed and tabulated, thus leading to a table similar to Table 68. For example, the average velocity in Tests *G, H, I, K,* and *L* for Ott meter No. 1, in Horizontal 1, would result from the sums of the velocities taken from the tests, Section 121, as follows: $(10.98 + 10.32 + 10.00 + 10.94 + 10.30) \div 20 = 2.627$. This figure, rounded off to 2.63, may then be entered in its proper place in Table 72.

TABLE 72.—MEAN INSTRUMENTAL VELOCITIES BY THE FIVE METERS IN THE FIVE HORIZONTALS, FOR TESTS *G, H, I, K,* AND *L.*

Meters.	Horizontal.				
	1	2	3	4	5
P	2.88	2.91	2.92	2.57	2.39
H_1	2.60	2.64	2.76	2.34	1.77
H_2	2.59	2.78	2.65	2.27	1.84
O_1	2.63	2.71	2.73	2.60	1.97
O_2	2.64	2.67	2.57	2.59	1.96
Average H	2.60	2.71	2.70	2.31	1.80
Average O	2.63	2.69	2.65	2.59	1.97

Just as Table 69 was compiled from Table 68 we may arrive at Table 73 for true average velocities in the horizontals during the five tests.

TABLE 73.—True Average Velocities Determined by Plates LXVII, LXVIII, LXIX, LXX, and LXXI, from Table 72.

Combination of meters.	Diagram.	Kind of oscillation.	Horizontal.				
			1	2	3	4	5
H-P	Plate LXX	Hor. and Vert.	2.87	2.90	2.91	2.57	2.28
H-P	" LXIX	Horizontal	2.82	2.88	2.88	2.52	2.22
O-P	" LXVIII	Hor. and Vert.	2.84	2.88	2.87	2.58?*	2.25
O-P	" LXVII	Horizontal	2.78	2.83	2.82	2.58?* (3.05)	2.20
O-H	" LXXI		2.67	2.70*	2.68*	3.10?	2.20
Mean of first four			2.828	2.872	2.870	2.562	2.238

* Price reads lower than Ott, or Ott reads lower than Haskell. Mean of meter velocities taken.

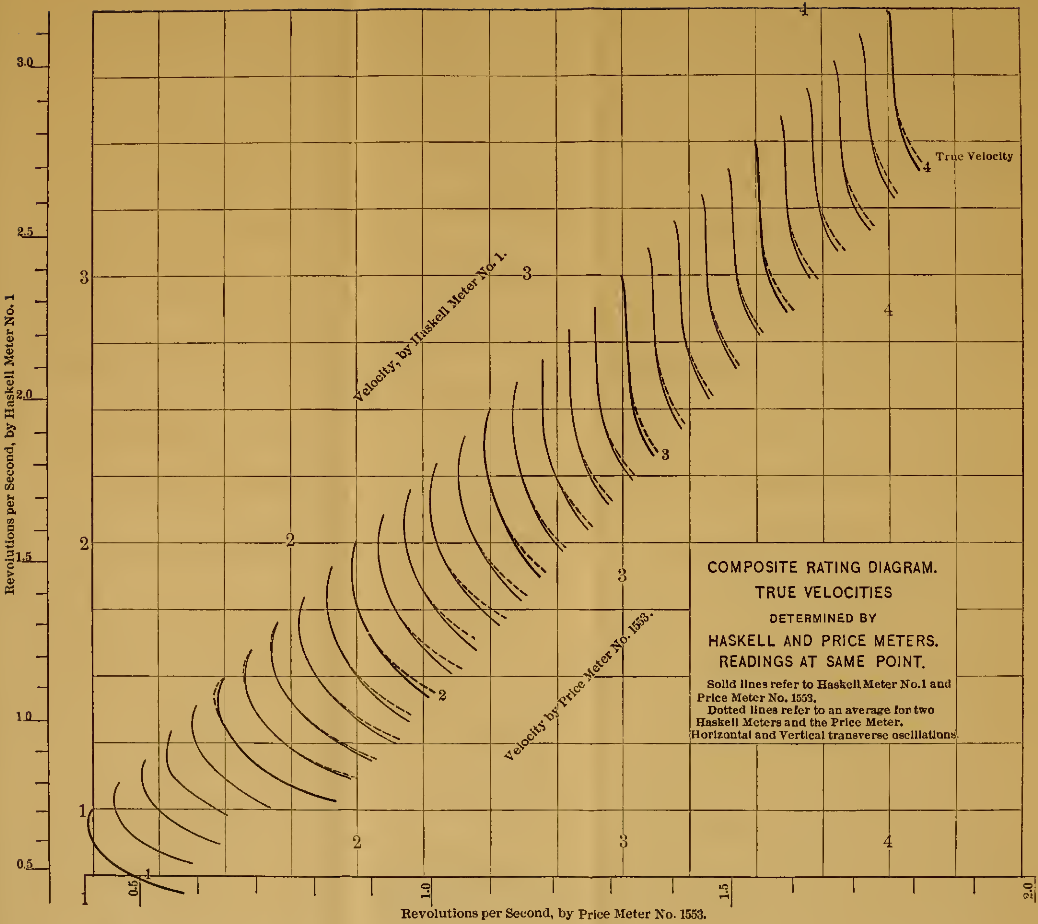
Continuing, as in Section 124, the factors of Table 74 have been computed.

TABLE 74.—Correction Factors for the Meters, Based on the Computed True Velocities for the Five Meter Horizontals in Tests *G, H, I, K,* and *L*, Determined by Plates LXVII, LXVIII, LXIX, LXX, and LXXI.

Meter.	Horizontal.					
	1	2	3	4	5	Mean.
Price	0.981	0.988	0.985	0.996	0.936	0.977
Haskell	1.089	1.060	1.062	1.112	1.240	1.113
Ott	1.074	1.068	1.082	0.988*	1.138	1.070
Mean						1.053

* This, of course, is abnormal. A further study would be required to eradicate the error.

Table 74 may then be used to calculate the sums of the velocities for each of the tests, noting that there is only one position in each test for a given meter in a particular horizontal, thus necessitating 25 multiplications to compute each test. We might have compared the meters for the intersection of each horizontal with each vertical position, thus obtaining 25 factors for the 25 meter rectangles in which each meter was operated, but this would have added considerably to the labor of the computation. Undoubtedly, a correction factor


COMPOSITE RATING DIAGRAM.
TRUE VELOCITIES
DETERMINED BY
HASKELL AND PRICE METERS.
READINGS AT SAME POINT.
Solid lines refer to Haskell Meter No.1 and Price Meter No. 1553.
Dotted lines refer to an average for two Haskell Meters and the Price Meter.
Horizontal and Vertical transverse oscillations.
Velocity, by Haskell Meter No. 1.
Velocity by Price Meter No. 1553.
True Velocity
Revolutions per Second, by Haskell Meter No. 1
Revolutions per Second, by Price Meter No. 1553.
3.0
2.5
2.0
1.5
1.0
0.5
1
2
3
4

for each of the 100 meter points in the head-race can be determined for each meter.

Table 75 illustrates the method in the computation of the sum of the velocities at the 100 meter points of Test *G*.

TABLE 75.—True Velocities for the 25 Intersections of Verticals and Horizontals in Test *G*, Based on the Factors of Table 74.
(Velocities, in feet per second.)

Horizontal.	Position and Meter.					
	$1 - O_1$	$2 - H_1$	$3 - P$	$4 - H_2$	$5 - O_2$	Total.
1	1 179	1 125	1 018	1 105	1 147	....
2	1 311	1 115	1 045	1 017	1 090	....
3	1 340	1 094	1 017	1 047	1 118	....
4	1 058	1 125	988	1 092	1 172	....
5	943	1 022	886	1 003	820	....
	5 831	5 481	4 954	5 264	5 347	268.77

The aggregates of velocity by this method are :
$G = 268.8$ ft. per sec.
$H = 268.3$ " "
$I = 270.4$ " "
$K = 261.0$ " "
$L = 267.8$ " "

126.—Computations for Tests G, H, I, K, and L, with Factors Based on Tests N, O, P, Q, and R.—The computations in Tables 76 to 83 have been made in a manner similar to those of the preceding two sections.

TABLE 76.—Mean Instrumental Velocities by the Five Meters in the Five Positions for Tests *N, O, P, Q*, and *R*.
(Compare Table 68; common discharge = 1 780 cu. ft. per sec. Common area = 619 sq. ft. Velocities, in feet per second.)

Meter.	Position.				
	1	2	3	4	5
Price,..............................	3.16	3.05	2.88	2.90	3.05
$Haskell_1$..........................	2.96	2.65	2.63	2.63	2.62
$Haskell_2$..........................	2.92	2.58	2.57	2.48	2.55
Ott_1..............................	2.95	2.79	2.67	2.71	2.65
Ott_2..............................	2.89	2.75	2.68	2.64	2.62
Average Haskell................	2.94	2.62	2.60	2.56	2.58
Average Ott......................	2.92	2.77	2.68	2.68	2.64

TABLE 77.—TRUE AVERAGE VELOCITIES DETERMINED BY PLATES LXVII, LXVIII, LXIX, AND LXX, FROM TABLE 76.

(Common discharge = 1 780 cu. ft. per sec. Common area = 619 sq. ft. Velocities, in feet per second.)

Combinations of meters.	Diagram.	Oscillations.	Position.				
			1	2	3	4	5
H-P...........	Plate LXX.......	Hor. and Vert.....	3.14	3.02	2.87	2.89	3.02
H-P...........	" LXIX	Horizontal	3.12	2.94	2.82	2.82	2.93
O-P	" LXVIII ...	Hor.and Vert	3.13	3.01	2.86	2.87	2.97
O-P...........	" LXVII	Horizontal	3.06	2.94	2.81	2.82	2.88
Means			3.11	2.98	2.84	2.85	2.95

TABLE 78.—CORRECTION FACTORS FOR THE METERS BASED ON THE COMPUTED TRUE VELOCITIES FOR THE FIVE METER POSITIONS IN TESTS *N, O, P, Q,* AND *R,* DETERMINED BY PLATES LXVII, LXVIII, LXIX AND LXX.

(Common discharge = 1 780 cu. ft. per sec. Common area = 619 sq. ft.)

Meter.	Position.					
	1	2	3	4	5	Mean.
Price	0.984	0.977	0.987	0.983	0.968	0.980
Haskell..............	1.058	1.138	1.092	1.113	1.143	1.109
Ott	1.065	1.076	1.060	1.063	1.118	1.076
Mean						1.055

TABLE 79.—AGGREGATES OF VELOCITY FOR TESTS *G, H, I, K,* AND *L* BY FACTORS BASED ON TESTS *N, O, P, Q,* AND *R.* METERS COMPARED BY POSITIONS.

Test.	Sum of velocities, in feet per second.
G..............	273.6
H..............	270.6
I..............	268.7
K..............	264.4
L..............	267.1

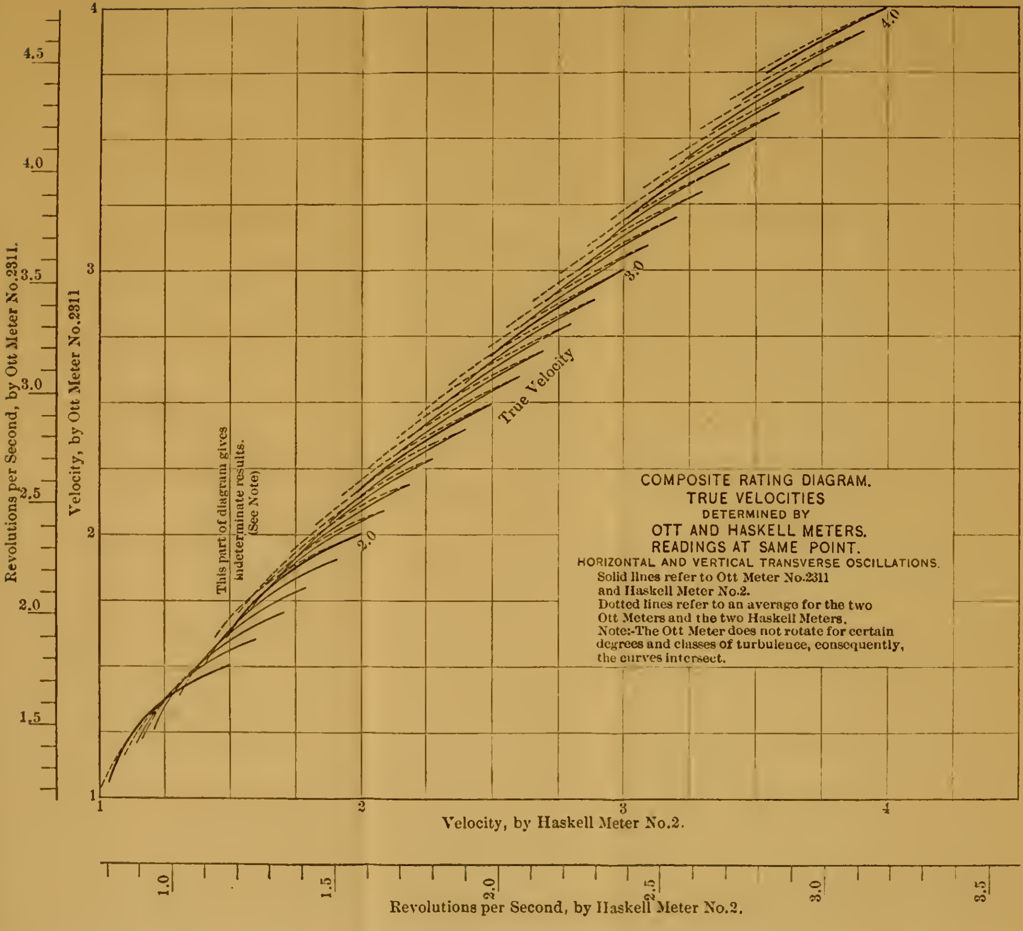

COMPOSITE RATING DIAGRAM.
TRUE VELOCITIES
DETERMINED BY
OTT AND HASKELL METERS.
READINGS AT SAME POINT.
HORIZONTAL AND VERTICAL TRANSVERSE OSCILLATIONS.
Solid lines refer to Ott Meter No.2311
and Haskell Meter No.2.
Dotted lines refer to an average for the two
Ott Meters and the two Haskell Meters.
Note:-The Ott Meter does not rotate for certain
degrees and classes of turbulence, consequently,
the curves intersect.
This part of diagram gives
indeterminate results.
(See Note)
True Velocity
2.0
3.0
4.0
Velocity, by Haskell Meter No.2.
Revolutions per Second, by Haskell Meter No.2.
Velocity, by Ott Meter No.2311
Revolutions per Second, by Ott Meter No.2311.
1
2
3
4
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5

TABLE 80.—MEAN INSTRUMENTAL VELOCITIES BY THE FIVE METERS IN THE FIVE HORIZONTALS FOR TESTS *N, O, P, Q,* AND *R.*

(Velocities, in feet per second.)

Meter.	Horizontal.				
	1	2	3	4	5
Price	3.16	3.08	3.18	2.94	2.66
Haskell$_1$	2.88	2.88	2.88	2.73	2.14
Haskell$_2$	2.82	2.80	2.75	2.64	2.11
Ott$_1$	2.94	2.87	2.89	2.79	2.29
Ott$_2$	2.93	2.82	2.91	2.74	2.22
Average Haskell	2.85	2.84	2.82	2.68	2.12
Average Ott	2.94	2.84	2.90	2.76	2.26

TABLE 81.—TRUE AVERAGE VELOCITIES DETERMINED BY PLATES LXVII, LXVIII, LXIX, LXX, AND LXXI, FROM TABLE 80.

(Velocities, in feet per second.)

Combination of meters.	Diagram.	Kind of oscillation.	Horizontal.				
			1	2	3	4	5
H-P	Plate LXX	Hor. and Vert	3.14	3.07	3.16	2.93	2.59
H-P	" LXIX	Horizontal	3.09	3.03	3.09	2.89	2.51
O-P	" LXVIII	Hor. and Vert	3.13	3.05	3.14	2.93	2.56
O-P	" LXVII	Horizontal	3.07	2.99	3.07	2.87	2.49
O-H	" LXXI		3.02	2.84	2.97	2.83	2.40
Average of first four			3.108	3.035	3.115	2.905	2.538

TABLE 82.—CORRECTION FACTORS FOR THE METERS, BASED ON THE COMPUTED TRUE VELOCITIES FOR THE FIVE METER HORIZONTALS IN TESTS *N, O, P, Q,* AND *R,* DETERMINED BY PLATES LXVII, LXVIII, LXIX, LXX, AND LXXI.

Meter.	Horizontal.					
	1	2	3	4	5	Mean.
Price	0.984	0.985	0.980	0.988	0.954	0.978
Haskell	1.090	1.068	1.105	1.084	1.197	1.109
Ott	1.057	1.068	1.074	1.053	1.123	1.075
Mean						1.054

TABLE 83.—AGGREGATES OF VELOCITY FOR TESTS *G, H, I, K,* AND *L,* BY FACTORS BASED ON TESTS *N, O, P, Q,* AND *R.* METERS COMPARED BY HORIZONTALS.

Test.	Sum of velocities, in feet per second.
G	269 3
H	269.2
I	271.8
K	261.6
L	268.3

127.—Discharges in Tests G, H, I, K, and L.—Four different velocity computations have now been made for Tests *G, H, I, K,* and *L.* It will be of value to compare these velocities before computing the discharges. Some idea can be gained thereby of the degree of precision attained by the current-meter observations and calculations. The comparison is made in Table 84.

TABLE 84.—AGGREGATE VELOCITIES AT THE 100 UNIFORMLY DISTRIBUTED POINTS IN THE HEAD-RACE OF UNIT NO. 13 FOR TESTS *G, H, I, K,* AND *L.* THESE TESTS WERE NEARLY IDENTICAL AS TO CONDITIONS OF OPERATION.

(Velocities, in feet per second.)

Nature of factors for correcting the meter records.	TEST				
	G	*H*	*I*	*K*	*L*
Based on average velocities in the five positions for Tests *G, H, I, K,* and *L,* reduced to common area and discharge	267.9	270.9	268.6	266.6	266.7
Based on average velocities by each meter in each horizontal for the five tests, *G, H, I, K,* and *L.* Not reduced to common area and discharge	268.8	268.3	270.4	261.0	267.8
Based on average velocities in the five positions for Tests *N, O, P, Q,* and *R,* roughly reduced to common area and discharge	273.6	270.6	268.7	264.4	267.1
Based on average velocities by each meter in each horizontal for the five tests, *N, O, P, Q,* and *R.* Not reduced to common area and discharge	269.3	269.2	271.8	261.6	268.3
Averages	269.9	269.8	269.9	263.4	267.5

By referring to Tables 70, 74, 78, and 82, it will be seen that the average values of the correction factors for all meters are, respectively, 1.054, 1.053, 1.055, and 1.054, among which the maximum variation is found to be not more than 0.2 per cent. As to an average value, the system of factors is very stable.

There are relatively larger variations, however, among the four estimates of the sum of velocities for any individual test. As a general rule, the sum of the velocities for any test is larger when based on factors determined from Tests *N, O, P, Q,* and *R* than when based on those determined from Tests *G, H, I, K,* and *L,* though, on the average, this discrepancy is not large, the total sum for the five tests for the four classes of factors differing only as in Table 85.

TABLE 85.—TOTAL SUM OF VELOCITIES FOR TESTS *G, H, I, K,* AND *L,* BY THE FOUR METHODS.

(Velocities, in feet per second.)

Factors based on tests:	Meters compared by:	Total sum of velocities, all five tests.
G, H, I, K, L	Positions	1 340.7
G, H, I, K, L	Horizontals	1 336.3
N, O, P, Q, R	Positions	1 344.4
N, O, P, Q, R	Horizontals	1 340.2

As to individual tests, it will be seen that there are maximum variations for *G, H, I, K,* and *L,* respectively, as follows: 2.1, 1.0, 1.2, 2.1, and 0.6 per cent. Thus, different computations of the sum of the velocities, and therefore of the discharge, may vary by 1 or 2 per cent. The best that can be done by the foregoing computations will be to take the average sum of the velocities for each test at the bottom of Table 84 for the calculation of the corresponding discharge.

Before making the calculation, it must be explained that there are two corrections to be applied to current-meter discharges before the best estimate can result.

128.—Pump Discharge Added to Current-Meter Discharge.—In Section 51 it was shown that the discharge of the centrifugal pump was about 900 gal. per min., or about 2 cu. ft. per sec. As the suction of the pump was up stream from the current meters, and the discharge was down stream, this 2 cu. ft. per sec. must be added to the discharge determined by the current meters in tests where the chemical and meter methods were run in parallel.

129.—Meter-Rack Correction.—It was suspected by the writer that the frame for supporting the meters had a retarding reaction on the speed of the meters, as the framework of the rack formed a considerable obstruction in the head-race.

Three comparative tests were run to determine the amount of retardation, but none was executed under satisfactory conditions, and reliable information cannot now be given concerning the effect in question. However, the tests will be described briefly.

The tendency of the meter rack is to cut off the flow through the obstructed portion of the race and divert it to the unobstructed portion. Therefore, a meter in front of the rack will tend to run slower, and one in the section above or below the rack will tend to run faster, than with the rack removed.

Accordingly, some arms were made for the purpose of attaching the meters to the rack in such a manner as to hold them exactly 2 ft. vertically under their usual positions in front of the rack. A traverse of the race with the meters attached to the arms was then made, after which a traverse was made with the meters in their usual positions. The results of the traverses show that the meters were retarded, but the tests were too limited to give an exact measure. One of the tests is probably excessive and one is deficient in velocity, the mean of the three tests probably being not far from the truth, though it would seem to the writer to be too small rather than too large.

TABLE 86.—Comparison of Velocities at Points in the Head-Race, When the Meter Supporting the Rack Is in the Usual Position, with Those at the Same Points When the Rack is 2 ft. Higher, the Meters Being Then Held on Arms Extending 2 ft. Below the Rack.

Meters on Arms.		Meters in Normal Position.	
Test.	Mean velocity, in feet per second.	Test.	Mean velocity, in feet per second.
24	2.494	25	2.450
71	2.559	70	2.601
72	2.690	73	2.668
Mean	2.581	Mean	2.573

On the average, there is an apparent retardation of 0.3%, due to the presence of the rack.

It may be explained that the meters always operated at the regular meter points in the race, the effect being that the rack was 2 ft. higher than its usual position when the meters were attached to the arms.

The method of suspending and operating the meters is explained fully in Part III.

130.—Computation of Discharge by Tests G, H, I, K, and L.—To obtain the metered discharges for these tests from the average sums of the velocities in Table 84, it is merely necessary to multiply the sums by the distribution factor and the resulting product by the area of the section. By applying additively about 7 cu. ft. per sec., the corrected discharge may be obtained, taking into account the pump and rack corrections. Table 87 results.

TABLE 87.—Discharge in Tests *G, H, I, K,* and *L*, Based on Average Aggregate Velocities Deduced by the Factors.

	Test.				
	G	*H*	*I*	*K*	*L*
Aggregate velocities in head-race, in feet per second	269.9	269.8	269.9	263.4	267.5
Distribution factor (tests of 1911)	0.00981	0.00981	0.00981	0.00981	0.00981
Mean velocity in head-race, in feet per second	2.648	2.647	2.648	2.583	2.623
Area of cross-section, in square feet	625.8	625.8	625.2	623.9	626.0
Discharge of meter section, in cubic feet per second	1 656	1 655	1 655	1 611	1 641
Discharge of centrifugal pump, in cubic feet per second	2	2	2	2	2
Correction due to rack error, in cubic feet per second	5	5	5	5	5
Total discharge, in cubic feet per second	1 663	1 662	1 662	1 618	1 648

131.—Discharge in Tests N, O, P, Q, and R.—As in the preceding section, the discharges for Tests *N, O, P, Q,* and *R* can be computed from the aggregates of velocity for these tests. Table 88 gives the aggregates for the four systems of factors deduced. The remainder of the calculations will be clearly understood without further remark.

132.—Discharge in Tests S, T, U, and V.—Tests *S, T, U,* and *V* have been computed in the same manner. The final results are shown in Tables 90 and 91.

133.—Comparison of Discharges by Current Meter with Those by Chemical Method.—In making this comparison it must be understood that the tests in which both methods were used were the earlier and less reliable ones, in which there is considerably more uncertainty than in the later (numbered tests following Test 25) series. The

TABLE 88.—AGGREGATES OF VELOCITY AT THE 100 UNIFORMLY DISTRIBUTED POINTS IN THE HEAD-RACE OF UNIT NO. 13 FOR TESTS *N, O, P, Q,* AND *R*. THESE TESTS WERE NEARLY IDENTICAL AS TO CONDITIONS OF OPERATION, BUT ARE NOT SO UNIFORM IN THIS RESPECT AS *G, H, I, K,* AND *L*.

Nature of Factors Used for Correcting the Meter Records.			Test.				
From tests:	Meters compared by:	Common area and discharge.	*N*	*O*	*P*	*Q*	*R*
G, H, I, K, L..	Positions......	625 sq ft.............. 1 660 cu. ft. per sec....	297.9	290.7	288.2	301.5	293.2
G, H, I, K, L..	Horizontals. ..	Not reduced...............	294.1	295.1	292.1	294.3	293.4
N, O, P, Q, R.	Positions......	619 sq ft.............. 1 780 cu. ft. per sec....	297.6	295.0	293.1	296.0	293.3
N, O, P, Q, R.	Horizontals. ..	Not reduced...............	294.5	295.5	292.7	294.7	294.0
Average values....			296.0	294.1	291.5	296.6	293.5

TABLE 89.—DISCHARGE IN TESTS *N, O, P, Q,* AND *R*, BASED ON AVERAGE AGGREGATE VELOCITIES DEDUCED BY THE FACTORS.

	Test.				
	N	*O*	*P*	*Q*	*R*
Aggregate velocity in head-race, in feet per second..............................	296.0	294.1	291.5	296.6	293.5
Distribution factor (tests of 1911)........	0.00981	0.00981	0.00981	0.00981	0.00981
Mean velocity in head-race, in feet per second............................	2.903	2.885	2.858	2.908	2.878
Area of cross-section, in square feet......	612.9	615.9	622.0	620.8	622.0
Discharge at meter section, in cubic feet per second..............................	1 778+	1 776	1 777+	1 805	1 790
Discharge of centrifugal pump, in cubic feet per second..........................	2	2	0	0	0
Correction due to rack error, in cubic feet per second..........................	5	5	5	5	5
Total discharge..........................	1 785	1 783	1 782	1 810	1 795

power was measured only by the switch-board instruments, and has been corrected as nearly as may be, but is probably not to be relied on to within 1 per cent. Hence, it is scarcely possible to state in all cases whether the current meters or the chemical method is the more accurate.

TABLE 90.—Aggregate of Velocity at the 100 Uniformly Distributed Points in the Head-Race of Unit No. 13 for Tests *S, T, U,* and *V*. These Tests were Run Under Conditions which are Variable from Test to Test.

Nature of Factors used for Correcting the Meter Record.			Test.			
From tests:	Meters compared by:	Common area and discharge.	*S*	*T*	*U*	*V*
G, H, I, K, L	Positions.	625 sq. ft. 1 660 cu. ft. per sec.	277.4	262.8	249.4	257.9
G, H, I, K, L	Horizontals.	Not reduced.	274.2	266.5	253.1	251.8
N, O, P, Q, R	Positions.	619 sq. ft. 1 780 cu. ft. per sec.	277.7	266.7	254.0	253.5
N, O, P, Q, R	Horizontals.	Not reduced.	274.6	267.0	253.6	252.5
Average value			276.0	265.8	252.5	253.9

TABLE 91.—Discharge in Tests *S, T, U,* and *V*, Based on Average Aggregate Velocities Deduced by the Factors.

	Test.			
	S	*T*	*U*	*V*
Aggregate velocity in head-race, in feet per second	276.0	265.8	252.5	253.9
Distribution factor (tests of 1911)	0.00981	0.00981	0.00981	0.00981
Mean velocity in head-race, in feet per second	270.6	260.7	247.6	248.9
Area of cross-section, in square feet	617.3	622.0	605.3	602.0
Discharge at meter section, in cubic feet per second	1 669	1 621	1 498	1 498
Discharge of centrifugal pump, in cubic feet per second	2	2	2	2
Correction due to rack error, in cubic feet per second	5	5	5	5
Total discharge, in cubic feet per second	1 676	1 628	1 505	1 505

In particular, Tests *T*, *U*, and *V* are very doubtful as to the chemical method, as it is practically certain that an error was made in the laboratory when titrating the salt solution samples of Tests *T* and *U*, and leaking pipes affected the results of Test *V*.

It appears that at this time the chemists had not succeeded in titrating all the samples of each test on the same day. Frequently, the salt solution samples were titrated on the day following the titration of the remainder of the samples, and, in the case of Tests *T* and *U*, these salt solution samples undoubtedly became interchanged, so as to cause Test *T* to give too great a discharge and Test *U* to give too small a discharge, each by several per cent.

In the case of Test *V*, Sample 22 titrated about 3 c.c. lower than it should, thus pointing to a leak in the corresponding pipe.

By interchanging the values of v as used in Tests *T* and *U*, Table 50, the discharges for these tests can be shown to be about 1 662 and 1 513 cu. ft. per sec., much more likely values than those given in the table. If Tests *T* and *U* are to enter in the comparison, these values should be used.

In the case of Test *V*, it is doubtful whether any improvement can be made in the calculation, further than to state that the discharge computed in Table 50 is too large, owing to the dilution of tail-water samples by reason of a leak. To compare methods, therefore, it will be advisable to omit Test *V* altogether.

Of course, the practice of titrating the salt solution samples at any other than the time at which the remaining samples of the same test are treated is contrary to the principles of group titrations, fully discussed in Section 48, Part II.

The comparison of the discharge by the two methods is given in Table 92.

TABLE 92.—Comparison of Discharges Determined by Current Meters with Those Determined by the Chemical Method.

Test.	Discharge Determined by:		Remarks.
	Current meter.	Chemical method.	
G	1 663	1 662	
H	1 662	1 662	The figures in parentheses are corrected from the erroneous results of Tests *T* and *U*, as explained in note at foot of Table 50. They are the only figures which should be used in these comparisons.
I	1 662	1 666	
K	1 618*	1 683	
L	1 648	1 647	
N	1 785	1 777	* Meter in fifth position appears to run slower than usual.
O	1 783	1 768	
S	1 676	1 640	
T	1 628	1 746 (1 662)	Meter rack jammed, delaying meter test, which ended 17 min. after chemical test.
U	1 505	1 440 (1 513)	
V	1 505	1 553	Chemical test untrustworthy. Sample 22 indicates a leak in sampling pipe.

Omitting Test *V*, the total discharges by current meter and chemical method are 16 630 and 16 680 cu. ft. per sec., respectively, results differing by 50 cu. ft., which is 0.3%, on the average, during the tests.

The close agreement between the two methods for several of the tests is more or less of a coincidence, but is significant of the fact that there are no large systematic errors in either method.

Of course, it will be understood that the current-meter computations would have been much simpler had the meter rack been arranged so that direct comparisons of two meters of different types could be made in one and the same test, rather than to have to resort to comparisons in different tests. However, the work is more interesting in its present form.

Before closing, it must be repeated that the tests in which comparisons of the results by meters and chemicals can be made are by far the least reliable of any, excepting possibly a few where leaks in the sampling pipes vitiated the results altogether. Such tests are limited to a few in the series numbered from 1 to 25.

For much valuable advice and assistance of various kinds the writer is greatly indebted to Messrs. T. J. Bostwick, I. G. Calderwood, John R. Freeman, P. H. Falter, E. E. Haskell, George K. Herzog, J. H. James, D. J. Jones, H. W. King, H. C. Sadler, I. B. Smith, E. W. Schoder, W. M. White, I. W. Wilson, A. S. Yount; and the engineers and chemists more or less directly connected with the tests: Messrs. C. E. Andrews, G. H. Bancroft, J. Earle Brennan, J. B. Campbell, Clyde L. Frear, Y. Habeshian, F. G. Hess, J. A. Lahey, W. A. Mudge, F. Nagler, A. J. Salathe, Arthur Schweier, C. M. Scudder, G. H. Spencer, M. L. Stack, K. T. Stearns, Thomas Sullivan, C. S. Thayer, D. C. Walser, W. H. Wagner, R. E. Ward, R. B. Whalen, A. L. Williams, and A. E. Wood.

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CURRENT METERS.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE ECONOMICAL TOP WIDTH OF NON-OVERFLOW DAMS

By William P. Creager, M. Am. Soc. C. E.

To be Presented January 5th, 1916.

Synopsis.

This paper treats of the most economical width of top of the commonly-accepted type of section of solid, gravity, non-overflow dams. The writer believes it to be the general opinion of engineers that the section with a zero top width, namely, a triangular section, contains the minimum area consistent with fixed assumptions; and that the adoption of a definite width of top for a roadway or other purpose is made at a sacrifice of economy. Presumably, for this reason, many dams have been built with tops as narrow as 5% of the height. This investigation, however, shows, that the most economical width of top, for usual designing assumptions, is not zero, but lies generally between 10 and 17% of the height, according to the assumptions used in the design. As the difference in the volumes of sections, having quite a wide range of top widths, is very small compared with the uncertainty of many of the designing assumptions, the writer feels that many of his readers may consider this paper of academic rather than of economic interest.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

It is just this point in particular, however, which the writer wishes to bring out, namely, that these investigations, as far as they have been carried, indicate:

1.—That there is little or no economy in the adoption of extremely narrow tops; and

2.—That exceptionally wide tops may be used, if desired, with comparatively little sacrifice of economy.

Assumptions.

The curves shown on Fig. 1 cover seven sections, designed under different assumptions. They indicate for each the most economical width of top, in terms of height, and the relative areas of sections having other top widths. The assumptions used have been designated by letters, and are as follows:

Location of Resultant.—

A.—Resultant, reservoir full, to intersect all horizontal joints at the exact extremity of the middle third; except near the top before the down-stream face departs from the perpendicular, where the resultant lies within the middle third.

B.—Same as *A*, except to intersect at a point within the middle third a distance equal to one-fifteenth of the width of the joint.

C.—Resultant, reservoir empty, to intersect all horizontal joints at the exact extremity of the middle third; except near the top, before the up-stream face departs from the perpendicular, where the resultant lies within the middle third.

D.—Resultant, reservoir empty, to have no influence on the design of the section.

Forces Considered.—

E.—Weight of concrete; assumed specific gravity, 2.25.

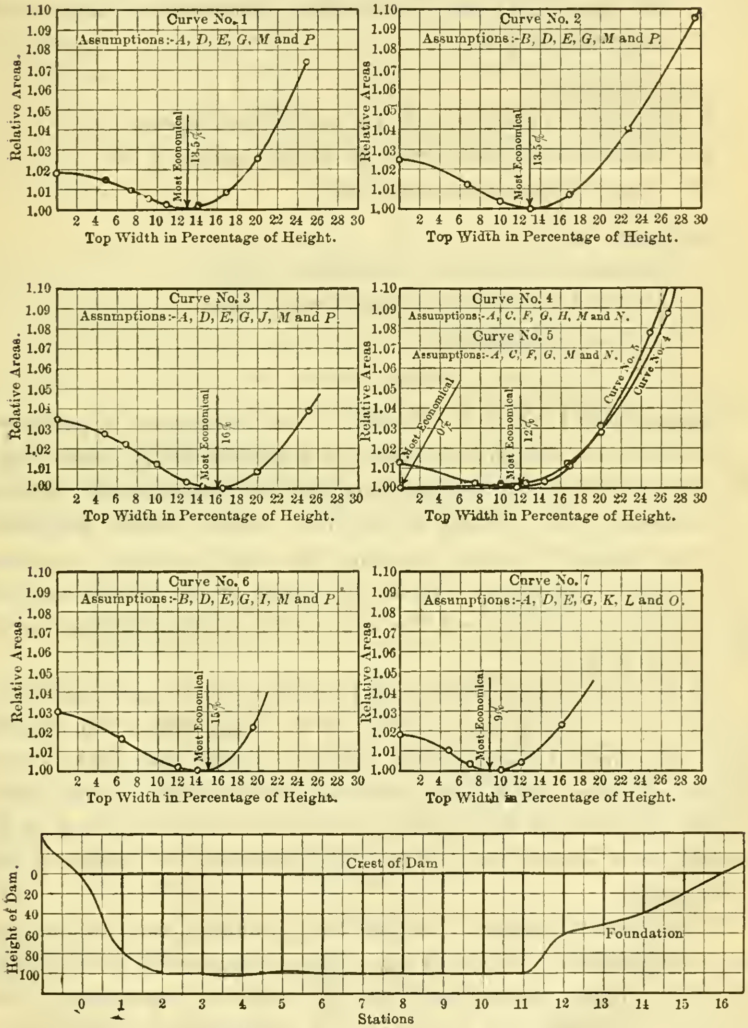
F.—Weight of concrete; assumed specific gravity, 2.33.

G.—Horizontal component of water pressures.

H.—Vertical component of water pressures on the battered up-stream face.

I.—Horizontal silt pressure; silt assumed to be a liquid with a specific gravity of 0.64 in addition to the water pressure. Depth of silt five-tenths of the height of the section.

ECONOMICAL WIDTH OF TOP OF NON-OVERFLOW DAMS.



PROFILE OF DAM SITE USED IN EXAMPLE.

FIG. 1.

J.—Uplift on all horizontal joints. Total uplift assumed to be represented by a triangle, the unit uplift equal to five-tenths of the hydrostatic pressure due to the total head of water at the up-stream side diminishing uniformly to zero at the down-stream side.

Joint Pressures.—

K.—Limited to 9 tons per sq. ft., at the down-stream face.

L.—Limited to 11 tons per sq. ft. at the up-stream face.

M.—Joint pressures not considered.

Up-stream Face.—

N.—Battered to conform to Condition *C* only.

O.—Battered to conform to Condition *L* only.

P.—Vertical throughout.

CURVE NO. 1.

Assumptions A, D, E, G, M, and P.—In starting the investigation of this subject, a design was made for a section 200 ft. high, with a top width of 10 ft. The assumptions used in the design are indicated by the letters. This section, or any part thereof, can be changed to suit other heights by simply changing the scale to which it is drawn, as all weights and forces vary as the square of the height, and both the moment of stability and the moment of overturning vary as the cube of the height.

All the dimensions of the top 40 ft. of this section were then multiplied by 2.5, resulting in a section 100 ft. high, with a top width of 25 ft. The area of the resulting section was then computed as being the area of a dam 100 ft. high, with a top width of 25% of the height.

The top 50 ft. of the original section was then multiplied by 2.0, resulting in a section also 100 ft. high, but with a top width of 20% of the height.

In this way a number of sections were produced, each 100 ft. high, but having different widths of top. The relative areas are plotted on Curve No. 1 as ordinates and the top widths in percentages of the height as abscissas. (A typical set of calculations is given in the Appendix.)

It will be noted from this curve that the most economical top width of sections designed in accordance with these assumptions, is about 13.5% of the height.

Curve No. 2.

Assumptions B, D, E, G, M, and P.—Curve No. 2 was prepared in the same manner and under the same assumptions as adopted for Curve No. 1, except that the resultant, reservoir full, instead of intersecting all horizontal joints at the exact extremity of the middle third (Assumption *A*), was required to lie within the middle third a distance of one-fifteenth of the width of the joint (Assumption *B*).

This curve, though slightly different in shape from Curve No. 1, also indicates the most economical width of top to be about 13.5% of the height.

Curve No. 3.

Assumptions A, D, E, G, J, M, and P.—Curve No. 3 was derived under the same assumptions as those governing the design of Curve No. 1, except that uplift (Assumption *J*) at all horizontal joints was included. The total uplift was assumed to be represented by a triangle, the unit uplift equal to five-tenths of the hydrostatic pressure due to the total head of water at the up-stream side diminishing uniformly to zero at the down-stream side.

This curve indicates the most economical width of top for these conditions to be about 16% of the height.

Curve No. 4.

Assumptions A, C, F, G, H, M, and N.—Curve No. 4 was prepared from a section, also designed in accordance with the assumptions used for Curve No. 1, except that in this case the specific gravity of the concrete was taken at 2.33 (Assumption *F*), instead of 2.25 (Assumption *E*), and the resultant was also required to intersect the base at the extremity of the middle third when the reservoir is empty (Assumption *C*). Assumption *C* necessitated a slightly battered up-stream face and the vertical component of the water pressure on this face was added to the forces acting (Assumption *H*).

In this case the most economical width of top appears to be about 12% of the height.

Curve No. 5.

Assumptions A, C, F, G, M, and N.—Curve No. 5 is the same as Curve No. 4, except that the vertical component of the water pressure on the battered up-stream face (Assumption *H*) was neglected. In this case, alone, the most economical width was found to be zero,

but the curve is seen to be nearly horizontal between 0% and 14%, at the latter width involving an increase of material of only ½ of 1 per cent. This curve will apply directly to "Theoretical Type No. 2," by Edward Wegmann, M. Am. Soc. C. E., as it will be noted that the assumptions covering the design of the section are exactly the same as those used by Mr. Wegmann. It might be remarked here that the condition of requiring the resultant to lie within the middle third with reservoir empty is often omitted by engineers, a vertical up-stream face being adopted, unless a batter is required for the condition of limiting toe pressures.

Curve No. 6.

Assumptions B, D, E, G, I, M, and P.—Curve No. 6 was based on the assumptions used in preparing Curve No. 2, except that in this case silt pressure (Assumption *I*) was included. The silt pressure was assumed to be a liquid with a specific gravity of 0.64 in addition to the water pressure, and its depth was assumed as five-tenths of the height of the section.

On account of the silt pressure, the expedient of changing the scale, resorted to in computing previous curves, would apply from the top of the dam to the surface of the silt only. The rest of each section had to be computed separately for each point on the curve.

Assumption *I* seems to lead to a slightly larger economical top width, appearing on the curve to be about 15% of the height.

Curve No. 7.

Assumptions A, D, E, G, K, L, and O.—Thus far there has not been taken into consideration the condition of limiting joint pressures (Assumptions *K* and *L*). Curve No. 7 is based on these assumptions. In other respects the assumptions used were the same as for Curve No. 1.

On account of Assumptions *K* and *L*, the expedient of changing the scale, resorted to in computing Curves Nos. 1 to 5, would apply only from the top of the dam to the elevation at which the limiting joint pressures began to govern the design. The rest of each section had to be computed separately for each point on the curve.

The vertical component of the water pressure on the battered up-stream face (Assumption *H*) was neglected in order to simplify

the calculations. A comparison of sections 200 ft. high was also used in computing Curve No. 7.

For this curve the most economical width was found to be about 9% of the height. In all probability, to include in the calculations the vertical component of the water pressure on the battered up-stream face would increase the most economical width, as it was seen from Curves Nos. 4 and 5 that it resulted in an increase from 0 to 12 per cent.

Method of Application.

It must be remembered that the curves apply only to dams of constant height throughout their length. In order to obtain the greatest economy, the top width, theoretically, should be a fixed percentage of the height at any point. As a varying width of top is objectionable, for many reasons, a constant width should be adopted which will be somewhat less than that corresponding to the most economical for the maximum height, the amount of such reduction depending on the relative quantity of material contained in that portion of the dam less in height than the maximum.

In order to indicate the amount of such reduction the writer has designed a dam for the profile indicated on Fig. 1 in accordance with the assumptions used in computing Curve No. 3, and found the most economical top width for this dam to be 14% as compared with 16% indicated on Curve No. 3 for the maximum section (100 ft.).

This indicates that very little reduction in top width is necessary unless the variation in height of dam at different points along the profile is considerable.

Conclusion.

The assumptions used herein cover in a general way most of the important conditions usually considered, with the exception of ice thrust. However, as the consideration of overturning forces in addition to the water pressure seems to increase the most economical top width, as in the case of uplift and silt conditions (Curves Nos. 3 and 6); and as the consideration of ice thrust, in itself, increases greatly the top part of the section, it seems logical to assume that an economical top width for ice thrust condition would be at least as great as that indicated in Curves Nos. 3 and 6.

It is believed, therefore, that, except for Curve No. 7 (which, however, would probably have been similar to the rest if the vertical

component of the water pressure had not been neglected), practically no economy results in selecting a top width for dams of practically uniform height less than about 14% of the height; and that, for some designing assumptions, a width of even 17% involves no sacrifice of economy.

It is true that the assumptions on which these conclusions are based do not consider sliding or vertical shear. It is believed, however, that cases where these considerations affect the shape of the section are the exception rather than the rule. Moreover, in the light of these investigations, as far as they have gone, it is hard to say whether these conditions would require a smaller or a larger top width than indicated in the curves.

The writer regrets that he has not had the time to carry these investigations farther.

APPENDIX

Calculations for Curve No. 3.

Assumptions A, D, E, G, J, M, and P.—

Let:

H = The height of the dam above any horizontal joint, the length of which is to be determined;

L = The length of a horizontal joint to be determined;

L_0 = The known length of the horizontal joint next above;

h = The vertical distance between these two horizontal joints;

w = The weight of 1 cu. ft. of masonry = 140.5 lb.;

A = The resultant weight of the dam, above the known horizontal joint, divided by w; it is also equal to the area;

m = The horizontal distance from the heel of the dam to the resultant A;

P = The total water pressure on the up-stream face of the dam divided by w;

U = The total uplift pressure on the unknown joint divided by w;

s = The specific gravity of the masonry = 2.25;

M_P = The moment, about the heel, of water pressure on the up-stream face of the dam divided by w;

M_U = The moment, about the heel, of the uplift divided by w;

M_A = The moment, about the heel, of the weight of the dam divided by w.

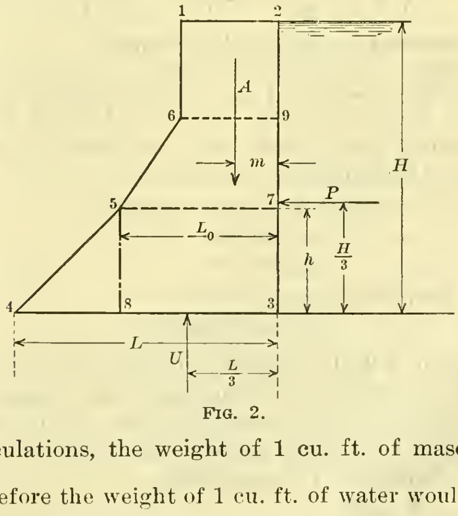


Fig. 2.

In the calculations, the weight of 1 cu. ft. of masonry was taken as unity; therefore the weight of 1 cu. ft. of water would be $\frac{1}{s} = \frac{1}{2.25}$.

The method of design, as worked out by Messrs. Morrison and Brodie, Wegmann, and others, was used.

A general formula to suit this particular case was first derived.

In Fig. 2 suppose the dam to have been already designed, down to the joint, 5-7. A general formula for the length, L, of the joint, 4-3, was derived as follows:

The weights and moments acting on the dam are given in Table 1.

TABLE 1.—Weights and Moments.

Portion.	Weight.	Moment about Point 3.
Masonry above joint 5–7.......	$+A$	$+A\,m$
Masonry in 5–7–3–8.............	$+L_0\,h$	$+\frac{L_0^2\,h}{2}$
Masonry in 4–5–8...............	$+\frac{L\,h}{2}-\frac{L_0\,h}{2}$	$+\frac{L^2\,h}{6}+\frac{L\,h\,L_0}{6}-\frac{L_0^2\,h}{3}$
Uplift, U......................	$-\frac{L\,H}{9}$	$-\frac{L^2\,H}{27}$
Horizontal water pressure, P..	Zero	$+\frac{H^3}{13.5}$

The total moment about Point 3 is:

$$A\,m+\frac{L_0^2\,h}{6}+\frac{L^2\,h}{6}+\frac{L\,h\,L_0}{6}-\frac{L^2\,H}{27}+\frac{H^3}{13.5}.$$

The reaction of the foundation is equal to the net weight of the dam, and the moment of the foundation reaction about Point 3 is the reaction multiplied by $\frac{2\,L}{3}$, or

$$\left(A+\frac{L_0\,h}{2}+\frac{L\,h}{2}-\frac{L\,H}{9}\right)\frac{2\,L}{3}.$$

As these two moments are equal, there results:

$$L^2+L\left(\frac{18\,A+4.5\,h\,L_0}{4.5\,h-H}\right)=\frac{2\,H^3+4.5\,L_0^2\,h+27\,A\,m}{4.5\,h-H}\quad\ldots.(1)$$

For a triangular dam 100 ft. high, the width of base was found from Equation (1), as follows:

Make $h = H = 100$ ft.

" $A = 0$

" $L_0 = 0$

Solving for H, there resulted:

$$H = 75.592 \text{ ft.}$$

Above the joint, 6-9, the resultant lies well within the middle third, and at the joint, 6-9, where the down-stream face begins to depart from the vertical, it is just at the extremity of the middle third. In order to find the height, 1-6, of this portion, the following assumptions for substitution in Equation (1) were made:

$A = 0$

$L_0 = L = 10$ ft., the width of top,

$h = H$

Solving for H, there resulted:

$$H = 13.229 \text{ ft.}$$

To find the length, L, of a joint 16 ft. from the top, the dam was assumed to have been designed, as indicated above, down to an elevation 13.229 ft. from the top. For this case:

$$\begin{aligned} L_0 &= 10 \\ A &= 13.229 \times 10 = 132.29 \\ m &= \frac{10}{2} = 5 \\ H &= 16 \\ h &= 16 - 13.229 = 2.771 \end{aligned}$$

With these substitutions in Equation (1), there resulted: $L = 11.10$ ft. The area and all other characteristics of the section from the top to an elevation 16 ft. from the top were then calculated. For the length of the joint 20 ft. from the top, the following substitutions in Equation (1) were made:

$$\left.\begin{aligned} L_0 &= 11.10 \\ A &= 161.52 \\ m &= 5.280 \end{aligned}\right\} \text{(from the foregoing)}$$

$$\begin{aligned} H &= 20 \\ h &= 20 - 16 = 4 \end{aligned}$$

With these substitutions, there resulted: $L = 13.06$ ft.

In this way, the whole section was designed by successive steps to an elevation 200 ft. from the top. A summary of the calculations is given in Table 2.

The next step was to derive from Table 2 a number of other sections, each 100 ft. high, but having different top widths. Columns 1 and 6 of Table 2 are repeated on Lines 1 and 2 of Table 3. A section of this type of any height can be adapted to any other height by simply changing the scale to which it is drawn, as the weights and water pressures are all functions of H^2, and the moments of the weight of masonry and water pressures are functions of H^3. This was done in Table 3. The heights and top widths indicated on Lines 1 and 3 were multiplied by the factors on Line 4, and entered on Lines 6 and 8, respectively. The new heights were all 100 ft.

The areas on Line 2 were multiplied by the square of the multiplying factor to give the new areas entered on Line 7.

The top width, as a percentage of the height given on Line 9, was obtained by dividing the quantities on Line 8 by the corresponding quantities on Line 6 and multiplying by 100 to obtain the percentages.

Calling the minimum area on Line 7 equal to 1.0, the other areas, in terms of this one, were found and entered on Line 10.

The quantities from Line 10, as ordinates, and the corresponding quantities from Line 9, as abscissas, were used in plotting Curve No. 3.

TABLE 2.—SUMMARY OF CALCULATIONS FOR A DAM 200 FT. HIGH AND A TOP WIDTH OF 10 FT. Assumptions *A*, *D*, *E*, *G*, *J*, *M*, and *P*.

(1) Height.	(2) P	(3) M_P	(4) U	(5) M_U	(6) A	(7) M_A	(8) L
13.229..........	38.890	171.49	14.70	49.00	132.29	661.45	10.00
16..............	56.888	303.40	19.73	73.00	161.52	815.76	11.10
20..............	88.888	592.59	29.02	126.26	209.84	1 108.2	13.06
25..............	138.88	1 157.4	44.81	240.92	282.81	1 642.6	16.13
30..............	200.00	2 000.0	65.47	428.83	372.24	2 444.9	19.64
40..............	355.55	4 740.7	121.64	1 109.7	607.29	5 231.7	27.37
50..............	555.55	9 259.2	196.98	2 327.9	921.42	10 193	35.46
60..............	800.00	16 000	290.27	4 211.8	1 316.2	18 018	43.54
70..............	1 088.8	25 407	401.57	6 911.0	1 792.1	29 367	51.63
80..............	1 422.2	37 925	529.87	10 528.5	2 348.3	44 862	59.61
100.............	2 222.2	74 074	836.89	21 014	3 697.6	90 582	75.32
150.............	5 000.0	250 000	1 889.8	71 428	8 415.3	316 172	113.77
200.............	8 888.8	592 592	3 359.5	169 299	15 029	756 664	151.18

TABLE 3.—DETERMINATION OF THE CO-ORDINATES USED IN PLOTTING CURVE No. 3.

1. Height from Table 2....................	30	40	50	60	70	80	100	150	200	100*
2. *A* " " 2....................	372.24	607.29	921.42	1 316.2	1 792.1	2 348.3	3 697.6	8 415.3	15 029	3 779.6
3. Top " " 2....................	10	10	10	10	10	10	10	10	10	0
4. Multiplying factor....................	$\frac{10}{3}$	$\frac{10}{4}$	$\frac{10}{5}$	$\frac{10}{6}$	$\frac{10}{7}$	$\frac{10}{8}$	$\frac{10}{10}$	$\frac{10}{15}$	$\frac{10}{20}$	$\frac{10}{10}$
5. " " squared..............	$\frac{100}{9}$	$\frac{100}{16}$	$\frac{100}{25}$	$\frac{100}{36}$	$\frac{100}{49}$	$\frac{100}{64}$	$\frac{100}{100}$	$\frac{100}{225}$	$\frac{100}{400}$	$\frac{100}{100}$
6. New height..........................	100	100	100	100	100	100	100	100	100	100
7. New *A*..........	4 136.0	3 795.5	3 685.6	3 656.3	3 657.4	3 668.0	3 697.6	3 740.1	3 757.4	3 779.6
8. New top..............................	33.33	25.00	20.00	16.67	14.29	12.50	10.00	6.67	5.00	0
9. New top width, in percentage of height.	33.33	25.00	20.00	16.67	14.29	12.50	10.00	6.67	5.00	0
10. Relative areas........................	1.134	1.0382	1.0081	1.0000	1.0004	1.0033	1.0114	1.0230	1.0277	1.0338

* Triangular dam.

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PAPERS AND DISCUSSIONS

WATER SUPPLY OF THE SAN FRANCISCO-OAKLAND METROPOLITAN DISTRICT

Discussion.*

By Messrs. Edwin Duryea, Jr., H. H. Wadsworth, and W. C. Hammatt.

Edwin Duryea, Jr.,† M. Am. Soc. C. E. (by letter).—On page 1536‡ Mr. Clement H. Miller states as follows: Mr. Duryea.

"* * *, the irrigation duty has been assumed [by the Geological Survey] at 2.5 acre-ft. of water [per year per acre] for the gross acreage of irrigable land, [the water being] measured at the point of diversion from the stream."

Mr. Miller also states, with reference to the report of the writer as Chief Engineer of the South San Joaquin Irrigation District:

"This report shows that, * * * the water used per acre per irrigation ranged from 5.8 to 15 in. (without waste), depending on the quality of the soil, a smaller quantity not being sufficient to flood the checks. The duty of water for seasonal irrigation is given as 1 sec-ft. for 80 acres, or 4 acre-ft. per acre, applied on the land from lateral ditches.

"Mr. Duryea's report is also based on actual known conditions covering both the Modesto and Turlock Irrigation Districts, and indicates a required duty of water 60% greater than that allowed in the report of the Geological Survey."

Mr. Miller is in error in drawing his conclusion from the so-called Duryea report. The 5.8 to 15 in. per irrigation quoted by him are specific measured instances of actual use; but the depth necessary depends largely on the skill and care with which the land has been prepared

* Discussion of the paper by H. T. Cory, M. Am. Soc. C. E., continued from October, 1915, *Proceedings*.

† San Francisco, Cal.

‡ *Proceedings*, Am. Soc. C. E., for August, 1915.

Mr. Duryea. for irrigation or "checked-up", and on the skill of the irrigator in applying the water; and the depth used too often has no relation to that needed, but, instead, is the maximum which can be obtained, even up to depths harmful to the land.

As stated in the abridgment,* the general conclusions of Mr. P. C. Berkefeldt's report to the writer are as follows:

"For fairly sandy soil and sandy soil, crop alfalfa * * *. For the average run of the land to be irrigated (*i. e.*, the average grade, length, and width of checks, etc.), it is best to use not under a 15-sec-ft. head for a time of about 20 minutes to a half hour per acre; depth of irrigation to be from 4 to 6 in., * * *."

As an average for all the lands in such districts as the Modesto, Turlock, and South San Joaquin, the advisable number of irrigations is believed to be six per year, which, combined with the depth of from 4 to 6 in. per irrigation, stated in the Berkefeldt report to be advisable for alfalfa, is equivalent to a depth per year on the net area irrigated of from 2 to 3 ft., measured on the land. As the result of all the writer's study in connection with the design of the South San Joaquin irrigation system, he decided that the proper average depth for the irrigation of alfalfa is 4.5 in. per month, or 2.25 ft. per year, measured on the net area irrigated.

To provide for a possible "peak use" of the water, and to meet demands for a very ample supply, the Supply Canal and the Main Distributary Canal of the South San Joaquin system were proportioned to deliver 6 in. net water per month on the net area to be irrigated, after all probable losses from absorption and seepage, and without encroachment on the safe "free-board" of the canal banks; and, to provide for "rotation", the Branch Canals were proportioned to deliver the 6 in. each month when in use only about two-thirds of the full time, and the District Ditches when in use only a few days during the month.

The 4.5 in. per month, or 2.25 ft. per year, measured on the net area, are for alfalfa alone. Hardly any other crops require as much water, and in nearly all large irrigated districts a considerable and increasing proportion of the total area is in other crops. Hence, in the writer's opinion, the advisable average use of water in such large irrigation districts should be materially less than 2.25 acre-ft. per year per acre of net area. This is as measured on the lands irrigated that year, presumably about 80% of the gross area of the irrigation district; referred to the gross area, the advisable depth would be materially less than $2.25 \times 0.80 = 1.80$ ft. per year.

Of course, to supply these net depths of water on the lands to be irrigated, enough additional water must be diverted from the river into the headworks of the system to provide for all absorption and

* *Engineering News*, September 11th, 1913, p. 503.

seepage losses from the canals between the river and the irrigated lands. In the design of the South San Joaquin irrigation system, it was concluded that, at the most, 36% of the water diverted from the river might be lost by absorption and seepage from the canals and ditches—that part lost from the Distribution System tending to supply sub-surface irrigation more or less, however, and hence being only a partial loss—and that 64% or more would be available for the direct surface irrigation of the land. Under this supposition, the corresponding depth measured at the diversion point would be (less than 1.80 ft. ÷ more than 0.64) = somewhat less than 2.81 ft. Mr. Duryea.

Although realizing that most irrigators will dissent from this conclusion, it is the writer's judgment that for such irrigation districts as are under discussion (where it is very desirable to keep the ground-water at least 6 ft. below the ground surface and usually difficult to do so on a large proportion of the district), an average irrigation depth of 2.5 ft. per year, referred to the gross area and measured at the diversion from the river, is advisable and not unreasonably low, and is sufficient to furnish adequate irrigation to lands which have been checked with reasonable skill and care, and when irrigated with reasonable skill.

The gross area of the South San Joaquin Irrigation District is 71 050 acres. In the writer's judgment, the necessary and advisable water supply is about 600 cu. ft. per sec., as measured at the diversion point from the river, or about 480 cu. ft. per sec., as measured at the edge of the District lands. Expressed in this way, the corresponding advisable "duties" of the water are about 118 and 148 acres per cu. ft. per sec., respectively.

To permit of building up reservoir storage, even while irrigating, the headworks, the canals, and the tunnels above the reservoir site were proportioned for much greater flows; and to permit of a possible "peak use" of the water and to meet desires for an ample water supply, the canal below the reservoir was proportioned to deliver 650 cu. ft. per sec. at the edge of the District, corresponding to a "duty" of about 110 acres per cu. ft. per sec. The gate outlets from the Branch Canals and District Ditches are proportioned to deliver water on the lands at a duty of 134 acres (gross area) per cu. ft. per sec.

The writer is informed by a skillful irrigator of the South San Joaquin District that (on land prepared with reasonable care by "border" or "strip" checking) he is able to irrigate alfalfa adequately on sandy land with a "head" of 16⅔ cu. ft. per sec., at the rate of 20 min. or somewhat less per acre. That head flowing for 16⅓ min. is equivalent to 4.5 in. of water on the net area.

The writer's remarks apply almost equally well to the discussion* by Rudolph W. Van Norden, M. Am. Soc. C. E.

* *Proceedings*, Am. Soc. C. E., for September, 1915.

Mr. Wadsworth. H. H. Wadsworth,* M. Am. Soc. C. E. (by letter).—This paper and the discussion on it have been read with great interest. On account of his connection with its preparation, the writer has noted with considerable satisfaction the generally expressed commendation of the report of the Advisory Board of Army Engineers to the Secretary of the Interior on the investigations relative to a water supply for San Francisco and the Bay communities.

On the appearance of Mr. Cory's paper, notes were made of several points on which the writer at first had aspirations of basing a discussion. He discovered, however, that with very few exceptions these points bore on the relative development of the Los Angeles and the San Francisco Bay regions; and it seemed wiser to him not to enter this controversy.

In considering the merits of the several distant sources of supply which have been proposed, the author mentions the necessity for caution in using the writer's figures. The writer does not question the propriety of using great caution in this respect, but suggests that a careful reading of his report will show that, as far as practicable, the unit costs applied to the City's Hetch Hetchy project were used in estimating costs of developing other proposed sources; and that successive installations were as nearly equal (in volume) for the different schemes as practicable, so that whether the estimated costs are or are not close approximations; relatively, they are so.

If, owing to the present cheapness of power derived from other sources, the value of hydro-electric power is much less than that used in the report mentioned, the present value of sources of water supply with possibilities of large power development relative to others with no such possibilities may be aggrandized. It can scarcely be doubted, however, that the ultimate (and probably not distant) value of such possible power plants will be substantially that of the capitalization of the market value of the power which they may produce.

As pointed out by Mr. Cory and also by Mr. Van Norden, the actual future need of water from the Tuolumne River for irrigation is uncertain. The writer wishes to call attention, however, to the fact that his estimate of such need is not so materially less than others, as would appear from Mr. Van Norden's discussion.†

As stated in his report transmitted to the Secretary of the Interior, and as quoted in the letter of the Director of the United States Geological Survey to the Secretary of the Interior in response to the Senate resolution mentioned by Mr. Van Norden, the writer has taken the duty of water for the Turlock and Modesto Districts at 2½ acre-ft. per acre of land actually irrigated, and has allowed 23% for evaporation and seepage losses. For the total irrigation need from the

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* San Francisco, Cal.

† *Proceedings*, Am. Soc. C. E., for September, 1915.

Tuolumne River, he applies this duty to an area 60% in excess of the aggregate area of the two districts and gets an annual need of 1 132 000 acre-ft., as against a maximum requirement for any one year of 1 042 043 acre-ft. for the Turlock and Modesto Districts, as reported by Mr. Burton Smith. When volumes of water as large as either of these come to be needed, the time will have arrived for introducing such forms of construction as will decrease waste; and actual losses should be reduced to a figure not very different from the writer's assumption. Mr. Wadsworth.

The differences pointed out by Mr. Van Norden in results reached by Dr. George Otis Smith, Chief of the Geological Survey, and by the writer, are due to differences in assignment of substantially the same total storage capacity to irrigation and to city water supply use, respectively, and to his (Dr. Smith's) allowing full estimated desirable irrigation use during the extremely dry periods like 1898-99 and 1912-13, at the expense of municipal use, rather than skimping the irrigation use (but not the acquired rights) over an area much in excess of that now in organized districts, or of that now using water for irrigation, so that a full municipal demand of 400 000 000 gal. daily may be supplied through such periods.

W. C. Hammatt,* M. Am. Soc. C. E. (by letter).—The writer is of the opinion that the attacks on the Hetch Hetchy project are a little ill-timed, particularly in view of the discussion by Mr. Van Norden.† Bonds have been voted for the fulfillment of the project, part of the necessary land has been acquired, the Government permit has been obtained, a compromise agreement satisfactory to both parties has been entered into between the City and the Irrigation Districts, and ratified by Congress in the form of the Raker Bill. Actual construction has commenced on the road and railroad work, as well as on the diversion tunnel preliminary to the construction of the dam. Mr. Hammatt.

At this stage of the proceedings, the only thing which would justify the re-opening of the question is a decided proof of the inadequacy of the water supply, or the impracticability of its utilization. In the absence of such decided proof, the criticism becomes purely destructive, and should be condemned as such. Has any such proof been brought forth? At the present stage, comparisons of the Hetch Hetchy project with the other proposed sources of supply are beside the mark, and only the arguments against the project itself need be considered.

The Freeman report was not solely the work of one engineer, but the collaboration and compilation of many, and no expense was spared to obtain all the possible data necessary to insure the accuracy of the conclusions reached. The subsequent report of the Board of

* San Francisco, Cal.

† *Proceedings*, Am. Soc. C. E., for September, 1915.

Mr. Hammatt. Army Engineers checked this report in its essential points. The only things to be considered, then, are the weight of the contradictions made by other engineers of the conclusions reached from these data.

First, in regard to the cost of the project: Mr. Van Norden has questioned the prices based on the construction cost of the Los Angeles Aqueduct. It is possible, and even probable, that his contentions are correct. It is also possible that changes in conditions of labor and materials between the time when the report was made and the time when the construction will be done will affect these prices still further. This, however, was a matter pertinent to the case of the comparison between the Hetch Hetchy and other available sources of supply, but is not a pertinent matter now that the decision has been made. If the work should cost more than the preliminary estimates (made solely for the purpose of determining the practicability of the project) showed, it will not be the first public work which has cost more than the estimate. Should the additional cost be sufficient to require the raising of additional funds, it will not be the first municipal project which has required an additional bond issue for its completion. The estimates of cost of the other projects submitted are subject to the same reservations.

Second, regarding the feasibility of the pressure tunnel, thrown into doubt by Mr. Van Norden: Apparently, he overlooked the fact that the major portion of the proposed tunnel is pressure in design only. The location of most of the tunnels is such that they carry only sufficient pressure to insure the necessary flow. In the language of the Freeman report (page 121):

"The aqueduct for substantially the entire distance down from the mountains to the edge of the San Joaquin Valley is proposed to be built of the pressure tunnel type, but with barely sufficient pressure to make sure that its section will always be filled under the various conditions of future use."

The exceptions to this are the power-drops, and although these were estimated as tunnels in the Freeman report, there is nothing to hinder their construction in the usual manner, should this be deemed advisable when the final details of the system are worked out. The power-drop tunnels are not essential parts of the project, and the details given in the preliminary report are a small matter on which to base a condemnation of the scheme.

Third, in regard to the sufficiency of the supply: One of the main points on which the opponents of the Hetch Hetchy project base their opposition is the question as to the adequacy of the flow of the Tuolumne River to furnish San Francisco with the necessary supply after deducting the needs of the irrigable lands in its catchment area. In the determination of this matter, the acreage is a known quantity, but the demand of water per acre irrigated is the point on which the

determination of the ultimate quantity needed hinges. One reason that the water required for the irrigation of a certain crop is so difficult of determination by comparison with the use on similar lands for which data are available, is that the use in such localities is generally based on the quantity of water available rather than on the actual quantity necessary. There is no doubt that where water is plentiful an excessive use has been made by irrigators, due to their desire to get all they are entitled to, regardless of the effect it may have on their own and near-by land. It is also a fact that much water has been wasted by inefficient methods of application. The data gleaned from systems under these conditions have given rise to much of the doubt as to the adequacy of the $2\frac{1}{2}$ acre-ft. per acre estimated as the ultimate irrigation need of the irrigable land under the flow of the Tuolumne River. There is, however, a constant tendency of irrigation systems, and a consequent constant decrease in the waste of water. Mr. Hammatt.

The writer had occasion to make very accurate determinations as to canal losses and as to the duty of water applied to the land under the San Joaquin and Kings River Canal and Irrigation Company's system extending over several years. The results of the investigations were as follows:

(1).—That 45% of the water taken into the system was lost by percolation and evaporation in the entire system.

(2).—That the quantity of water measured as flowing to the land for irrigation was 2.44 acre-ft. per acre for 1907, 1.49 acre-ft. for 1908, and 1.44 acre-ft. for 1909.

The drop in the quantity used in 1908 below that used in 1907 was due to the company's change in irrigation charge from an acreage basis to a quantity basis. Prior to 1908, the rate was made per acre irrigated, regardless of the quantity of water used thereon, but, in that year, the rates were changed to so much per second-foot for 24 hours. This change was made for two reasons: First, because the extraordinary demand for water was forcing the system beyond its capacity; and second, because the excessive use was drowning the land. The result of the change was fully up to expectations, as the irrigators, when they were obliged to pay for the water used, reduced their use to their needs. Although the company has since returned to the acreage basis of charges, the educational work has been lasting, and the demand still remains as it was during the years when the rates were fixed on the quantity basis. The duty previously given was for all crops, the prevailing crop, however, being alfalfa, about 70% of the land being irrigated for this crop.

The writer is very familiar with the lands on the east side of the San Joaquin Valley from the Stanislaus River to Kings River, and

Mr. Hammatt.

has made several examinations of irrigation systems in this area. From his knowledge of the conditions in this territory, he is able to deduce the following conclusions:

(1).—That the canal losses in the Turlock and Modesto Irrigation Districts would be considerably less than those under the San Joaquin and Kings River Canal and Irrigation Company's canal system, due to the lesser canal length and greater fall of the former. The losses from seepage and evaporation in the canals of the Modesto and Turlock Irrigation Districts would undoubtedly fall below 35% of the total intake.

(2).—That the necessary use of water on the land in these districts would be at least as small as that under the San Joaquin and Kings River Canal and Irrigation Company's system, as the soil evaporation would be less and the sub-surface drainage probably no greater. This conclusion is deduced from an intimate knowledge of the soil conditions in both localities.

The necessary use of water, then, as measured at the intake, would be, considering 1.5 acre-ft. per acre necessary on the land itself, $1.5 \div 0.65 = 2.31$ acre-ft. per acre. Or, if the gross acreage is considered, as about 20% of the land is occupied by roads, buildings, yards, and other non-irrigated tracts, the necessary quantity for the whole area would be 80% of 2.31, or 1.85 acre-ft. per acre. It would seem, therefore, that the allowance of 2.5 acre-ft. per acre referred to in the other discussions would be ample to meet all contingencies of waste due to canal regulation, individual excessive use, and whatever other extraordinary condition might arise. Although there is little question that a slightly greater quantity than this has been used in the past on these lands, it is apparent that the quantity used has been in excess of the needs, as is shown by the rise of groundwater and the drowning of the crops in many localities.

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PAPERS AND DISCUSSIONS

THE TWELFTH STREET TRAFFICWAY VIADUCT, KANSAS CITY, MISSOURI

Discussion.*

By L. R. Ash, M. Am. Soc. C. E.

L. R. Ash,† M. Am. Soc. C. E. (by letter).—There are few cities in the United States where the freight-house and manufacturing districts are so sharply isolated from the business center as in Kansas City. In addition to this division between the sections of Kansas City, Mo., Kansas City, Kans., with a population of nearly 100 000 and its large and varied manufacturing interests, is also separated from the business center of Kansas City, Mo., by the bluffs which extend along the north and west sides of the city. The traffic between these districts is extremely heavy and, for a number of years, it has been seriously handicapped by bad grades and indirect routes. Mr. Ash.

The apparent need of a trafficway between Kansas City, Mo., and Kansas City, Kans., and to the West Bottoms, was great enough to enlist private capital in the enterprise of building the Sixth Street or Inter-city Viaduct some 8 or 10 years ago. This structure was built as a toll viaduct, and although it has proved to be a failure, from a financial standpoint, this failure in no way indicates that such a structure was not needed, but demonstrates the reluctance of the American public to pay tolls. During the times that the Inter-city Viaduct was open for traffic without charge, the structure was crowded with travel, both to and from Kansas City, Kans., as well as to and from the West Bottoms.

The question of building a viaduct at 12th Street, connecting with the West Bottoms, has been discussed for the last 15 years, and although every one realized its great need, it seemed almost impossible

* Discussion of the paper by E. E. Howard, M. Am. Soc. C. E., continued from October, 1915, *Proceedings*.

† Kansas City, Mo.

Mr. Ash. to harmonize the various interests which in one way or another were affected by the building of the structure. The old viaduct at 12th Street provided only for street-railway traffic. All vehicular traffic between the West Bottoms and Kansas City, Kans., and the business center of Kansas City, Mo., had no thoroughfare from 6th Street on the north to 24th Street on the south, a distance of more than 1½ miles. When, in addition to this, we consider the fact that neither Sixth Street nor 24th Street led in the direction of the terminus for this large traffic, we see more clearly the great need of a trafficway leading to the heart of the business portion of the city.

Numerous meetings were held with representatives of the various organizations, such as the Team Holders Association, Real Estate Board, Commercial Club, etc., in an attempt to determine the most practicable structure, both as regards the requirements of traffic and the elimination of damages. These discussions were frequently very acrimonious, and politics cut no small figure in the handling of the proposition. The management of the whole enterprise was a shining example of the need of a more responsible and centralized authority in municipal affairs.

One of the great problems which confronted the promoters of the 12th Street Viaduct was that of grades. About 5% seemed to be the only practicable grade on 12th Street, and, in the minds of a great many people, this grade was prohibitive when heavy vehicular traffic was considered. The grade, as originally established, was 5.09%, and this would have necessitated a cut of about 17 ft. at the top of the hill, which would have resulted in a very heavy property damage estimated at about $250 000. This matter came up while the writer was City Engineer of Kansas City, Mo., and he suggested the elimination of this property damage by raising the grade to within about 4 ft. of the established grade at Washington Street. This increased the viaduct grade to 5.52%, a very slight change from the old one; and the opposition to the structure, thus avoided, abundantly justified the change.

The advent of the motor truck has decreased the necessity for lower grades, except where reasonably attainable, but, as a concession to the demand for a low-grade trafficway between the Bottoms and the business section, a lower deck was provided on the viaduct with a maximum grade of about 2½ per cent. It is expected to utilize this portion of the viaduct by building roadways along the bluff leading to the north and to the south from the east end of this lower deck. In the writer's opinion, the building of these roadways will not be justified, as they will lead to the old trafficway at 6th Street on the north, or to 17th Street on the south, neither of which points is in the line of travel, and from which there will be bad grades and other conditions not inviting to heavy vehicular traffic. It would seem that

Mr. Ash.

a better solution of the problem would be to tunnel from the east end of the lower deck to about 14th and Wyandotte Streets, thus insuring a very easy grade and a direct route for the distribution of the traffic from the West Bottoms.

Now that the structure has been completed and thrown open to the public, the great need of it is proved by the very great traffic both vehicular and by street cars, using it; and this is drawn to the structure in spite of the fact that the approaches to the viaduct have not been paved, as they await the settlement of the embankments at each end. The structure saves several minutes to each of the many people who go each day from the eastern portion of the city to the West Bottoms, and the 5.52% grade has proved to be no serious handicap, either to motor or horse-drawn vehicles.

It is interesting to note that the building of the viaduct has resulted in enhancing real estate values and rentals on East 12th Street and the adjacent streets for a distance of 3 or 4 miles eastward from the structure. This is because the opening of 12th Street provides a direct way to the West Bottoms for the large number of people employed there, without having to transfer or travel several blocks out of their way. The 12th Street cars, although greatly increased in number, are crowded with passengers, and the building of the viaduct has fixed 12th Street as the principal east and west artery of the city.

Mr. Howard has gone very fully into the design and construction features, and there is no additional comment to be made except to remark on the very satisfactory result, as shown by the finished structure. The surface finish of the concrete is not all that could be desired, but it is believed that it is in keeping with the character of the structure and, when conditions in the neighborhood are considered, it is thought that the more expensive methods of treating concrete surfaces would not have been justified in this case.

The specifications as drawn would permit the engineer to require the contractor to put a cement-gun surface on all columns and cantilevers, as well as on the outside surfaces of the longitudinal girders, but, in the writer's opinion, the attempt to place this finish on surfaces of the character in this structure, would not result satisfactorily, and it is believed that the surface as produced by the forms, with the rough places smoothed off with hammer and chisel and in certain places filled with mortar, has been the best treatment practicable.

The treatment of concrete surfaces is receiving a great deal of attention, and various methods have been advanced for producing a surface which is smooth and, at the same time, does not present the expressionless effect of a smooth mortar surface. The writer believes that specifications can most easily provide for the finish of concrete

Mr. Ash surfaces by requiring the bidders to tender on two or three different kinds of treatment at a stated price per unit of surface, leaving the engineer to choose that method which, in his opinion, most nearly suits the condition when the forms are removed. Of course, all specifications should require forms to be built of smooth lumber, with tight joints and true to line.

The character of surface desired should be determined by the general characteristics of the structure, its location, type of construction, etc. No attempt should be made to give the concrete the appearance of stone or other material, and it is thought that, with a few exceptions, a reasonably smooth surface which is produced by good forms and well-spaded concrete, is better than can be secured by special treatments.

On the 12th Street Viaduct 22 ft. of the upper deck is set aside for street-car tracks, and a roadway, 30 ft. wide on the south side, is left for vehicles, a sidewalk 5 ft. wide extending along the south side of the structure. Although this arrangement has some advantages over that of placing the street railway in the center of the roadway, it would seem that the space between the hand-rails would be used more efficiently with the tracks in the center, as is the case in an ordinary city street. About the only justification for placing the tracks in a space independent of the roadway, is the length of the structure and the grade. It is believed that a sidewalk, 8 ft. wide, and a clear roadway of about 50 ft. between the curbs, with street-railway tracks in the center, would have resulted in a structure which would have cost somewhat less and would have been better suited to the general traffic conditions. As it is, vehicles on the north side of 12th Street, at each end of the viaduct, must cross the street railway tracks.

The use of wood blocks for paving the roadway portion has proved entirely satisfactory. It was feared that the grade would cause the pavement to be slippery, but the blocks were laid with a lath between each row, and the remaining space was well filled with broken stone. The blocks are 3 in. wide, and the resulting surface undoubtedly provides a better footing for horses than stone block, asphalt, or brick. The writer has observed heavily loaded teams turning out of the street railway portions of certain streets where there is stone block pavement, to the wood block pavement at the sides, because of the better footing secured on the latter.

The laying of the wood blocks on the viaduct without a sand cushion has been entirely satisfactory, and demonstrates the wisdom of omitting one of the great sources of trouble with pavements of this type. With care in finishing the concrete base, the surface can be brought to true crown and grade without serious trouble, and

when this is done there is no justification whatever for the use of a sand cushion. Mr. Ash.

The structure represents a great amount of careful study of the architectural treatment as well as the engineering features, and it is becoming more and more apparent that engineers should give consideration to matters pertaining to the appearance of structures of this character. There seems to be little excuse for absolutely neglecting the esthetic treatment of engineering structures, and although the Profession has been prone to do this, it is gratifying to note a marked change in its attitude in this matter.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE ACTION OF WATER UNDER DAMS

Discussion.*

By W. R. Baldwin-Wiseman, Esq.

W. R. Baldwin-Wiseman,† Esq. (by letter).—On page 1304‡ the author has summarized the work of a few of the more important investigators of this subject; but, on a subject about which there are such extreme variations of opinion, it would have been better to have given a more detailed list. Schlichter§ supplied a much more complete one, and the writer has given fuller details in the three following papers: "The Flow of Underground Waters,"|| "Statistical and Experimental Data on Filtration,"¶ and "The Influence of Pressure and Porosity on the Motion of Subsurface Water."** Incidentally, another aspect of the subject is dealt with in "The Influence of the Thickness of the Pipe Wall on the Rate of Discharge of Water from Minute Orifices Piercing the Pipe."†† Mr. Baldwin-Wiseman.

A very careful investigation of the subject has also been undertaken, and full references to other work are given, by J. Versluys in "Le Principe du Mouvement des Eaux Souterraines."‡‡

The subject has been dealt with by the writer in "Statistical and Experimental Data on Filtration" (quoted previously), more particularly on pages 20 to 40, 55 to 61, and 127 to 130.

Data in the writer's possession, but as yet unpublished, show that the rate of interstitial flow and of loss of pressure are largely dependent

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* Discussion of the paper by J. B. T. Colman, Assoc. M. Am. Soc. C. E., continued from October, 1915, *Proceedings*.

† Southampton, England.

‡ *Proceedings*, Am. Soc. C. E., for August, 1915.

§ U. S. Geol. Survey, 19th Annual Report.

|| *Minutes of Proceedings*, Inst. C. E., Vol. CXLV, 1905–06, pp. 309–352.

¶ *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXI, 1909–10, pp. 15–157.

** *Quarterly Journal*, Geol. Soc., Vol. LXIII, 1907, pp. 80–105.

†† *Transactions*, Assoc. Water Eng., Vol. XII, 1907, pp. 373–380.

‡‡ Published in Amsterdam in 1912.

Mr. Baldwin-Wiseman

on the material of the sand grains, on their angles—that is, whether they were formed by subaerial or submarine denudation—and on the relative homogeneity of the sand mass; and the results of experiments conform more closely to an arbitrary law when the sand grains composing a mass are of fairly uniform size than when the constituents of the mass are heterogeneous, even though the mean diameter of the grains of the mass, when determined by a size analysis, may be almost identical with that of the uniform or graded sand. This point is well illustrated in Fig. 2 of the paper "Statistical and Experimental Data on Filtration". In that paper the writer has endeavored to represent these factors of roughness, angularity, etc., by a term which (for lack of a better one) has been called the thickness of the ideal film, not because such a film exists, but because it is a facile conception, and, as will be seen by consulting the table on page 33 of that paper, sands with practically the same porosity have ideal films, varying in the ratio 1 : 0, 1 : 7 and 0 : 4. So much for the velocity of interstitial flow. From unpublished investigations the writer has found that the greatest loss of pressure occurs in interstitial passages which have extreme and rapid change in cross-section.

The writer concurs entirely with Conclusion 1, on page 1324,* as it is in agreement with conclusions succinctly stated on page 319 of his paper "The Flow of Underground Water",† and are well illustrated in the diagrams, *a* and *b*, on the same page; and they are fully confirmed by later experiments (not yet published) with graded and natural sands similar to those described in "Statistical and Experimental Data on Filtration".‡

The writer also agrees with the substance of Conclusions 2, 3, and 4, as his own work has led him to similar conclusions, but he cannot agree with the deductions of Conclusion 6, for reasons already detailed herein. This, however, is doubtless due to the method of arriving at the effective size and uniformity coefficient, a subject on which the writer has already expressed an opinion and indicated better methods in his paper on filtration.

* *Proceedings*, Am. Soc. C. E., for August, 1915.

† *Minutes of Proceedings*, Inst. C. E., Vol. CLXV.

‡ *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXI.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

CONCRETE-LINED OIL-STORAGE RESERVOIRS IN CALIFORNIA: CONSTRUCTION METHODS AND COST DATA

Discussion.*

By C. P. Bowie, Assoc. M. Am. Soc. C. E.

C. P. Bowie,† Assoc. M. Am. Soc. C. E. (by letter).—The writer is much interested in this paper. He notes that the roof of the reservoir described was covered with roofing paper, on which was placed a layer of asphaltum and gravel. It has been his experience that such coverings, for receptacles used for the storage of oil, are very unsatisfactory and, at best, cannot be properly considered more than temporary. Mr. Bowie.

The average roofing paper is made by immersing an ordinary deadening felt (or similar material) in a solution of hot asphaltum and then passing the saturated sheet through a set of heavy rolls. In such a treatment the asphaltum not only adds to the quality of the body of the paper by toughening it, but also renders it, to a certain degree, impervious to water. The more volatile hydro-carbon compounds contained in crude oil pass off at comparatively low temperatures in the form of vapor, and their effect on these roofing papers is to "cut", as it were, the saturating material, so that in a short time the fibers of the paper, especially of that portion covering the cracks between the sheeting boards, are held together so loosely that they can be ruptured with ease by the pressure of the finger. It follows, then, that such roofings soon offer almost no resistance to the passage of gas formed by evaporation.

* This discussion (of the paper by E. D. Cole, Assoc. M. Am. Soc. C. E., published in August, 1915, *Proceedings*, and presented at the meeting of September 15th, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† San Francisco, Cal.

Mr. Bowie. In the writer's opinion the proper covering for a storage reservoir, especially one in which crude oil for refining is to be kept, should be of steel plates, and "gas-tight". For this purpose, $\frac{1}{8}$-in. plates, which can be had in sizes 48 in. wide and in lengths up to 120 in., fastened together with $\frac{5}{16}$-in. rivets with $1\frac{1}{2}$-in. pitch, are ideal. In lieu of caulking, a thread weave (about $1\frac{1}{4}$ in. wide) which has previously been immersed in red lead, is placed between the laps of the sheets before riveting.

Near the center or highest point of such a roof, outlet pipes are placed, through which the gas can be drawn off, to be condensed or otherwise disposed of.

Regarding the loss by evaporation and the value of the product lost, to quote from the paper:

"A reservoir having a capacity of 750 000 bbl. loses 5% of oil per year, 1% of which is due to evaporation and 4% to seepage. In this case, 4% loss, at a value of 40 cents per bbl., represents an actual cash loss of $12 000 per year."

Assuming the losses by evaporation as stated by Mr. Cole to be correct; 1% of 750 000 bbl. would be 7 500 bbl., which product would be a material somewhat lighter than gasoline, probably between a gasoline stock and casing head gas, and worth at the least consideration about 10 cents per gal., or $4.20 per bbl. The annual loss due to evaporation then would be $31 500 as compared with $12 000 lost by seepage; from which it would appear that an adequate covering for a reservoir is of more value than a concrete lining. This, of course, applies primarily to crudes which are to be used for refining purposes; however, considering the ever-increasing demand for gasoline and other refined products, it may be a matter of only a short time before necessity will require that only oils which have at least been topped, shall be sold for fuel purposes.

It is estimated that a steel roof for a reservoir costs about $2\frac{2}{3}$ times as much as the present type of wooden roof covered with paper. Returning again to Mr. Cole's figures, this would make the cost per barrel for roof, say, 8 cents, as compared with 3 cents; or an increase of 5 cents per bbl. of capacity. The saving effected by the recovery of the oil which otherwise would be lost by evaporation, would then pay for such a roof in less than $1\frac{1}{2}$ years.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE ASTORIA TUNNEL UNDER THE EAST RIVER FOR GAS DISTRIBUTION IN NEW YORK CITY

Discussion.*

By Messrs. F. Lavis, Milton H. Freeman, W. H. Bradley, James F. Sanborn, Edward Wegmann, R. C. Kellogg, and W. W. Brush.

F. Lavis,† M. Am. Soc. C. E. (by letter).—During the past few years there has been a notable development, and almost entirely in the vicinity of New York City, in the construction of tunnels at such depths below water level as to preclude the possibility of using air pressure to prevent the inflow of water, should seams in the rock afford it an opportunity to reach the tunnel with the full head due to the depth below the water surface. Mr. Lavis.

The other tunnels of this nature which have been built, have been along the line of the Catskill Aqueduct, where the restrictions confining the location within certain definite limits were not so rigid as in the case of the Astoria Tunnel, and, therefore, it was possible to fix them with greater regard for the avoidance of probable bad geological conditions. In the case of the deep siphons of the Aqueduct, sufficient diamond drill borings were taken to enable competent geologists to predict fairly successfully the character of the rock, and the location was made in such a way that the tunnel would lie wholly within the zone in which it was expected to find sound rock, free from faults, or to cross the latter at or nearly at right angles. So far as the writer knows, these expectations were reasonably well fulfilled, and although grouting of water-bearing seams was resorted to, and very high pressures were used, there has not been recorded, thus far,

* This discussion (of the paper by John Vipond Davies, M. Am. Soc. C. E., published in August, 1915, *Proceedings*, and presented at the meeting of October 6th, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

Mr. Lavis. any experience comparable to that encountered in the Astoria Tunnel, and so well and fully described in this paper. The latter is unique in describing methods of driving a tunnel through unstable ground with seams of decomposed rock having direct connection with navigable waters, at a depth of more than 100 ft. below the lowest depth at which work could be carried on under air pressure, the recovery of the tunnel after complete flooding, and the solid grouting of the whole tunnel section to form a stable material through which the tunnel was finally driven.

It seems to the writer that it is out of the question to discuss in much detail the methods described in the paper; we have had no similar experience with which to compare it—the situation was unique, the methods were effective, and the job is finished. Some other scheme might have been effective, but no one can say positively that it would, the only possibility known to the writer is freezing, and there is nothing in past experience to lead one to believe that it would have been better, or so good. Probably it is only effective in fairly homogeneous material softer than rock, and probably would not work at all in the shattered rock encountered in this tunnel.

The Society, as well as the author, is to be congratulated on the excellent manner in which the paper has been presented, and the completeness of the description contained therein. The writer believes that his paper,* describing the construction of the Bergen Hill Tunnels of the Pennsylvania Railroad, contained the first tabulated statements in reference to any tunneling work in North America, showing number of feet drilled per cubic yard, quantities of explosives, actual drilling time, etc., though, in a lesser degree, such records had been published in regard to some of the Alpine tunnel work in Europe. He has noted, therefore, with considerable satisfaction, the presentation of similar data, not only in this paper, but in several other descriptions of tunneling operations in recent years. The statement of progress in number of cubic yards per shift or per day is far more definite than the statement of the number of feet of advance which may or may not be of the full clean section. The statement showing the complete items of the plant is most useful, as are also the details, so often omitted, of the smaller items, in regard to the excavation of the rock, such as quality of steel, sizes of holes, kind of fuse, batteries, etc.

The construction of the concrete arch to hold the roof in bad ground before building the side-walls or invert is not new in this tunnel, although it is a comparatively recent development in the art of soft-ground tunneling. So far as the writer knows its first important use was in the Providence Tunnel of the New York, New Haven and Hartford Railroad, built about 10 years ago.

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* *Transactions,* Am. Soc. C. E., Vol. LXVIII, p. 84.

Mr. Lavis.

The question as to the desirability or otherwise of using dry rock packing to fill the space between the normal section of the lining and the actual excavation is an interesting one. In this case, as well as in nearly all cases of the tunnels on the Catskill Aqueduct, tight concrete packing and thorough grouting were clearly indicated as the only effective method of surely preventing the least disturbance by settlement of the surrounding ground. In cases similar to that of the Bergen Hill Tunnels of the Pennsylvania Railroad, however, the use of dry rock packing over the arch was not only far less expensive, but proved to be most effective in allowing the ground-water to reach the drainage pipes at the sides and thus enter the side drains in the floor, providing a dry tunnel with very little water-proofing.

The value of this paper and the importance of the work which it describes can only be fully understood by a realization of the fact that it marks a distinct step forward in the art of tunneling. For ages, almost, tunneling was only possible through fairly sound rock, or materials which could be held by close timbering. The shield and the use of compressed air, brought into really general use only within the last 30 years, although they had been used previous to that time, enable tunnels to be driven through water-bearing materials to depths of about 100 ft. below the surface of the water, the pressure at greater depths being so great as practically to prohibit effective work. We now have a method of driving tunnels at almost any depth in water-bearing ground, and, within certain limits, through almost any class of material.

Perhaps it is not without interest at this time, when it has been suggested that mining engineers should be called in to advise as to methods of subway construction, to note—without in the least detracting from the ability of mining engineers and miners in their particular field—that nearly all the developments in the art of tunneling through difficult ground, beyond the art of timbering in comparatively small sections, have been made by civil engineers, and for transportation purposes, rather than by mining engineers or miners. In the really vast tunneling operations which have been carried on in New York City and its vicinity in recent years, it is undoubtedly within the bounds of truth to state that not only has this work been the greatest, most extensive, and most difficult of its kind, that is, of tunneling, ever undertaken and carried out successfully anywhere, but it has been done with fewer accidents, and all the engineers responsible for this state of affairs deserve commendation.

The record of this work on the Astoria Tunnel shows, also, as do all records of successful advances in the art of tunneling, and nearly all records in recent years of advance in the rate of progress, the absolute necessity of such preliminary study, careful planning, and efficient organization as usually only engineers are trained to carry

Mr. Lavis. out. It must be remembered, however, that though a carefully studied plan must be worked out and adhered to, engineers engaged in this kind of work must be able to adapt it to all sorts of variations as they arise. It is far from easy, amid the noise, dirt, partial darkness, and, oftentimes, apparent confusion underground, to carry out a plan which looks well on a drawing-board in a comfortable, well-lighted office; and when added to all this, the whole East River is likely to drop in on one at any minute, plans are likely to go by the board.

Mr. Freeman. MILTON H. FREEMAN,* Assoc. M. Am. Soc. C. E. (by letter).—One of the most interesting features of this paper is the description of the method of excavating through decomposed water-bearing rock. An excellent opportunity was afforded to obtain positive data concerning the results of the grouting, as the subsequent excavation exposed the grout-filled seams. In work of a similar character on the Catskill Aqueduct, the following were found to be essential features:

1.—An impervious bulkhead against which to grout, which also furnishes an anchorage for the grout pipes;

2.—A grout tank pressure considerably above the rock water pressure against which grout is injected;

3.—Adapting the consistency of the grout to the size of the cavity to be filled, using thicker grout for very large seams, and very thin grout for the fine seams.

On the Catskill Aqueduct the first attempt to fill water-bearing seams ahead of the excavation was made at Shaft 4 of the Rondout Siphon. This difficult piece of work is fully described by John P. Hogan, Assoc. M. Am. Soc. C. E., in his paper entitled "Sinking a Wet Shaft".†

This shaft passes through water-bearing strata, 131 ft. thick. Nipples were swedged into drill holes, valves put on, and the drilling was continued through the pipe and valve into the softer water-bearing strata, the holes furnishing channels for grouting the rock seams. The method was quite similar to the procedure in the Astoria Tunnel. It was not found necessary to place a concrete blanket in the bottom of the shaft as a bulkhead against which to grout, though, at one time, the rock was so soft that considerable grout was wasted in clogging the surface seams. Altogether, 2 960 bags of cement were injected, and grout tank pressures up to 275 lb. per sq. in. were used, giving a net pressure over and above the ground-water of from 100 to 180 lb. per sq. in. Large seams were especially well filled, the largest one being 8 in. wide, and quantities of water, up to 400 gal. per min., were stopped at different groutings. As excavation progressed, however, there was considerable seepage from the smaller seams; this was

* Brooklyn, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 398.

not cut off, and, for the entire depth of the shaft, amounted to about 450 gal. per min. Mr. Freeman.

At many other places along the line of the Aqueduct, the leakage from clean-cut open seams was stopped entirely. Near the east shaft of the Hudson River Siphon, an open seam discharging about 500 gal. per min. was completely dried by placing a concrete bulkhead across the heading and forcing in neat cement grout under a pressure of 700 lb. per sq. in., which was 200 lb. greater than the ground-water pressure.

It was common practice in permanent shafts, which were concrete-lined, to place this lining—at least, across water-bearing strata—during the process of shaft-sinking, putting in weep-pipes to vent the water, which pipes were grouted after the concrete had hardened. This method proved effectual in shutting off leaks of considerable size; for instance, a leak of 125 gal. per min. in Shaft 5 of the Rondout Siphon was completely shut off in this manner.

The grouting of the pressure tunnels proper involved one consideration which was not found in the case of the shafts, or in the case of any tunnel which, during use, is subject to normal air pressure. The lining in such a shaft or tunnel is in compression from the pressure of the surrounding rock, earth, or ground-water. The pressure tunnels of the Catskill System pass deep under the valleys, far below the hydraulic gradient, and will be operated under an outward pressure of from 150 to 650 lb. per sq. in.; although the ground-water pressure relieves much of this, there remain net bursting pressures up to 200 lb. per sq. in. under operation. Consequently, the total net outward pressure is great in the case of tunnels ranging from 11 to 14½ ft. in finished diameter. The concrete lining of such tunnels is not thick enough to hold under tension, and depends on the rock backing to carry the load. Seamy, water-carrying rock furnishes a ready channel to take away water which may escape through the concrete lining. Furthermore, rock which is very seamy and broken does not furnish such a good backing for the concrete lining. Thus, the grout which was injected into the surrounding rock seams, helped in two ways: in solidifying the rock and making a better backing for the concrete, and in closing some channels which would carry away any water that might leak through the concrete lining. To meet this condition, an attempt were made, by using the tunnel lining as a bulkhead, to grout farther back into the seams, and to fill smaller seams than had been done during the process of excavation.

The first trial was made at Shaft 4, north, of the Rondout Siphon, where the tunnel passes through the water-bearing High Falls shale and Binnewater sandstone, both of which are broken, porous rocks, particularly so at this point, as they are twisted and crushed in a fault zone. The length of tunnel in this rock is about 250 ft.

Mr. Freeman. During excavation an effort was made to grout off the leakage in the heading, and a little was stopped, but the attempt was not wholly successful. No concrete bulkhead was built against which to grout, and the rock was so weak that the face could not be solidified sufficiently to form a backing for grouting pressures. Finally, a test-hole was driven with a diamond drill 250 ft. ahead through the water-bearing strata. Gradually, the leakage decreased, indicating a lowering of the ground-water level, more pumps were put in, and excavation was completed without additional grouting. The maximum leakage in this section was 2 000 gal. per min. A complete account of this work has been given by B. H. Wait, Assoc. M. Am. Soc. C. E.*

The thickness of the concrete lining through this wet section was 24 in., twice the ordinary minimum thickness. As a drip-pan to protect the soft concrete from the incoming water, and as a form of reinforcement for the tunnel lining, a steel shell was erected for a length of 174 ft. This consisted of 6-in. I-beam ribs bent nearly to a circle, but open at one side, the open ends fitting to the concrete invert which was placed first. For a covering, $\frac{3}{16}$-in. plates were bolted to the ribs, and no attempt was made to secure water-tight joints. The concrete invert contained a longitudinal drain with Y-branches to each side, which picked up the leakage. The tunnel at this point was on a 15% incline, and drained itself readily.

The space between the steel shell and the rock was very carefully dry-packed, and afterward the concrete lining was placed inside this shell. Two sets of grout pipes were used, one leading through the concrete to this dry-packed section, the other set being carried from 1 to 3 ft. into the rock seams.

The dry packing outside the concrete was grouted first, while the pipes to the seams remained open to discharge the rock leakage, except those which were closed temporarily on account of leaking grout, when some of the thick grout worked back into the larger seams. Before grouting the dry packing, the inward leakage was 900 gal. per min., but after this grouting was completed, the deep-seated pipes discharged only 540 gal. per min., as the thick grout had worked back into some of the larger seams and cut down the leakage.

A start was made to grout the rock seams proper by injecting an ordinary mixture (95 lb. of cement to $5\frac{1}{2}$ gal. of water) of neat grout into a deep-seated pipe leaking about 4 gal. per min. and 3 bags of cement plugged the seam. Then, at the suggestion of T. H. Wiggin, M. Am. Soc. C. E., Designing Engineer of the Pressure Tunnels, this pipe was re-opened, cleaned out, and a very thin mixture used ($23\frac{3}{4}$ lb. of cement to 28 gal. of water). Of the latter mixture, 24 bags were injected into this same hole which had refused at 3 bags of the thicker

* *Engineering Record*, Vol. LXIII, p. 660, June 17th, 1911.

Mr. Freeman.

mixture. The use of the very thin grout was continued with grouting pressures up to 300 lb. per sq. in. The tunnel-service pressure was 100 lb. per sq. in., higher pressures being furnished by a Westinghouse (booster) compressor. Two compressors were piped with two grout tanks, so that either compressor could be used with either tank. With one tank charging and the other discharging, the stream of grout was kept practically continuous. The rock-water pressure grouted against was 20 lb. per sq. in. If a pipe took grout easily at a pressure of 100 lb., the mixture was thickened a little (47½ lb. of cement to 28 gal. of water), and, if this continued for some time, the thickening was increased to 1 bag per 28 gal. of water. When the injection pressure began to rise, the mixture was thinned again. Pipes were considered full which refused the thinnest grout under a pressure of 300 lb. It was noticeable that the grout traveled some distance (from 30 to 40 ft.) in the seamy rock, showing up at other pipes, and cutting down the leakage at those pipes and filling the seams back of them. Not more than one in ten of the pipes leading to the rock seams took any considerable quantity of grout, though every pipe was tried. After all grouting was completed in this section, the inward leakage was less than 1 gal. per min., due, perhaps, to a good concrete lining and not wholly to grouting. In a length of 174 ft. of tunnel, 940 bags of cement were injected into the rock seams, the finished diameter of the tunnel at this place being 14½ ft.

After the ground-water level was restored, four test holes were drilled about 3 ft. out into the grouted rock, where heavy leakages had occurred at the time of excavation. Two of these holes were practically dry, the third discharged ½ gal. per min., but the fourth struck a water-bearing seam and discharged 71 gal. per min. The latter hole was in the bottom of the tunnel, the others were at the horizontal diameter and the top. It is conceivable that the bottom might show the poorest results, as the ultimate direction of grout flow is upward. A gauge reading showed the rock water pressure to be 117 lb. per sq. in.

Another section of the same siphon, through blocky, seamy Helderberg limestone, took 946 bags of thin grout in a length of 50 ft. Two test holes, 3 ft. into the rock, showed no leakage.

Though absolute assurance cannot be given that all the seams were filled, the volume of cement injected indicates that many of them were closed by this method, which thicker grout would have clogged at the openings and left unfilled.

A difficulty arose in grouting a combination of large and small seams, as the large seams furnished an easy egress for thin grout, and the small seams clogged at the entrance with the thicker mixture. By increasing the cement per batch, if the pressure of injection

Mr. Freeman. remained low, and decreasing it again as the pressure rose, an attempt was made to adjust the consistency of the batch to the size of the seam.

Mr. Bradley. W. H. BRADLEY, ESQ.*—At a time when the outlook on the tunnel work was very dark, a prominent official of the Company said to the speaker "I was informed by an engineer that the tunnel is a failure—that it will never be completed". The speaker replied "You have never heard me say that, when you do you may take it to be a fact; at present it is not a fact".

To carry Mr. Davies' description of the pipe testing a little farther: About 2 weeks ago, one of the main pipes in the tunnel was tested, first by filling it with water which was allowed to rise about 40 ft. in the stand-pipes, giving a pressure of about 20 lb.; the joints were then inspected thoroughly for possible leaks, after which the water was run out and an air pressure of 40 lb. per sq. in. was applied, each joint being inspected by applying soap and water. After this inspection, water was again put in, the stand-pipes being filled to the top of the shaft, which created a pressure of about 125 lb. per sq. in. in the pipe. This may seem to have been unnecessary—after the previous tests—but it was a precautionary measure intended to expel all air from the pipe, because air and gas form an explosive mixture and there is likely to be serious trouble if a light reaches it. When the pipe was filled with water, the gas valves at both ends were opened, and, as the water was drawn out, the main was filled with gas with no mixture of air. This main is now in daily use.

The same procedure is being followed with the second main.

The head-houses are now being erected over the tunnel shafts, the elevators, pumping plant, telephone and lighting circuits are being installed, and the work will soon be completed.

Mr. Sanborn. JAMES F. SANBORN,† ASSOC. M. AM. SOC. C. E.—It would be interesting to have a description of the difficulties in putting in the vertical pipes in the shaft. Were any curving rings or shims required? The speaker asks for this information because, in the work of the New York Board of Water Supply, there was trouble in erecting the risers of the City tunnel shafts, which were steel pipes, and it was found expedient to use a hub and spigot arrangement.

Mr. Wegmann. EDWARD WEGMANN,† M. AM. SOC. C. E.—This paper is a very valuable contribution to the engineering literature of tunneling, and, doubtless, will be consulted by other engineers who encounter difficulties similar to those met in the Astoria Tunnel. The paper is very complete, as it describes in detail, not only the construction of the tunnel, but, also, the plant and methods used.

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* Chief Engineer, Astoria Light, Heat and Power Company, New York City.

† New York City.

The speaker is particularly interested in the paper because he was one of the engineers on the tunnel, constructed in 1885 to 1891 under the Harlem River as part of the New Croton Aqueduct, in driving which decomposed rock, similar to that described by the author, had been encountered. The tunnel was about 1 300 ft. long and required a circular excavation, 14 ft. in diameter. Mr. Wegmann.

Before the work was begun, numerous diamond-drill borings were made to ascertain the character of the rock through which the tunnel was to be driven. Near Manhattan Island the rock was found to be gneiss, but for the greater part it was limestone having many seams. A pocket of decomposed rock was discovered in the gneiss formation, and was investigated by making borings about 50 ft. apart.

Based on the indications of the diamond-drill borings, the tunnel was planned so as to have its invert about 155 ft. below mean tide. It was thought that at this level there would be at least 30 ft. of solid rock above the tunnel, but when the heading had advanced about 300 ft. from Manhattan Island, a seam of rock, full of water, was struck by the drills. Water poured into the tunnel, and the workmen ran for their lives. Sand and stone soon filled the drill holes, and made it possible to close them with wooden plugs.

A diamond drill was taken into the heading, and the ground in front and below the heading was explored by borings. In order to insure safety while this work was going on, a strong timber bulkhead, with a suitable gate, was built in the heading, just back of the diamond drill.

The test borings showed that the seam of decomposed rock extended about 75 ft. below the tunnel, instead of terminating 30 ft. above it, as had been believed by the engineer. In front of the heading, the seam was found to be 28 ft. wide, and beyond it there was limestone containing seams in which the water was under the full pressure due to the river, *i. e.*, under a head of more than 150 ft.

The important question arose as to how the tunnel was to be constructed through the seam of decomposed rock and through the limestone. Negotiations were held with the company controlling the patents for the freezing process, which had been used very successfully for sinking shafts through quicksand. This company was very willing to undertake the work on a percentage basis, but would not give any guaranties with regard to success.

The engineers in charge of the work were anxious to try to drive a pilot tunnel through the soft ground, but the contractor who was to do this work was very loath to undertake it, as it necessitated buying large pumps. Apparently, he had more influence with the Aqueduct Commissioners, under whose direction the New Croton Aqueduct was built, than the engineers of the Commission. By an order of this Commission, the engineers were directed to abandon the 300 ft.

Mr. Wegmann. of tunnel that had been excavated to the soft seam, and to start another heading about 307 ft. below mean tide. At this level no difficulties were encountered.

The only additional cost of constructing the tunnel at the new level was in the lowering and hoisting of materials, but this was more than offset by the fact that the tunnel was very dry at the lower level. In spite of these circumstances, the contractor, assisted by able counsel, sued the City of New York for seventy items, aggregating about $350 000, for the additional cost of constructing the tunnel at the lower level, and for some other extras. The original contract price for the work was only about $500 000.

The lawsuit against the City was begun in 1890, but the contractor's lawyers waited 16 years, before they brought the case to trial. They thought, probably, that their chances of success would be better when the engineers and inspectors, who had been connected with the work, had gone West or died. The City won the suit twice, but the Court of Appeals sent it back a third time for re-trial. The case was about finished before a referee, when this gentleman, unfortunately, died. Twenty-four years after the suit was begun, it was settled out of Court by the City paying the contractor $25 000 on claims amounting to $350 000. It would be interesting to know what part of the $25 000 was received by the contractor's lawyers and how much by the contractor himself. The speaker mentions this lawsuit to show some of the difficulties, other than soft seams, which engineers encounter in tunneling.

Mr. Kellogg. R. C. KELLOGG,* ASSOC. M. AM. SOC. C. E.—The Astoria Tunnel is one of the principal connecting links in a vast scheme of gas distribution which comprehends sending out from the Astoria works, at some future time, approximately 200 000 000 cu. ft. of gas every 24 hours, part of which may eventually be pumped as far as Albany. This will not only rid Manhattan Island of unsightly gas manufacturing structures, but will also benefit the towns through which the supply may travel. The Astoria gas plant itself, when completed according to the plans conceived, will contain ten units, each capable of manufacturing 20 000 000 cu. ft. of gas every 24 hours, and ten storage holders each having a capacity of 15 000 000 cu. ft.

Mr. Brush. W. W. BRUSH,† M. AM. SOC. C. E.—It may be of interest to mention the fact that the engineers of the New York Board of Water Supply studied a line for a water supply tunnel, which would follow somewhat closely that of the gas tunnel the construction of which has been described so clearly and interestingly by Mr. Davies. The future water supply of New York City will necessitate the delivery of large quan-

* Brooklyn, N. Y.

† New York City.

tities into Queens Borough, which contains the maximum area available for development within the city, and where eventually an enormous manufacturing center will probably be found, with a large consumption of water. Mr. Brush.

The studies made several years ago to locate the tunnel to deliver the Catskill supply to the five boroughs indicated that a rock tunnel could probably be driven successfully into Astoria and carried through the rock into the Borough of Brooklyn, but that the cost would be decidedly greater than a tunnel along the adopted line under the Bronx and Manhattan to Brooklyn. One of the uncertain elements of cost was the driving of the tunnel under the East River, probably passing under Randall's and Ward's Islands. The question of determining the character of the rock at the contacts between the various formations was deemed to be a difficult problem.

Ultimately, however, in all probability, a tunnel will be driven under the East River, near the Gas Company's tunnel, for the purpose of delivering an additional water supply to the Borough of Queens and in part to the Borough of Brooklyn. The experience of the Gas Company will then furnish especially interesting and valuable information for the engineers who will lay out the line and grade of the later tunnel. A water tunnel can readily be driven at any depth desired, and therefore the engineers may avoid some of the difficulties mentioned by Mr. Davies.

The riser pipes in the shafts of the Astoria-Bronx tunnel are embedded in concrete, and are therefore self-supporting.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

INDUCED CURRENTS OF FLUIDS
Discussion.*

By Messrs. Clemens Herschel and Carl George de Laval.

Clemens Herschel,† M. Am. Soc. C. E. (by letter).—A special case of induced currents of fluids is shown by the currents flowing into the throat of a Venturi tube from the outside, when thus allowed to flow in by suitable orifices. The writer has experimented on such currents on an unusually large scale, these experiments being described in a paper entitled "The Fall-Increaser", in the *Harvard Engineering Journal* of June, 1908. This is an account of experiments made to establish the proper proportions of an apparatus called the "fall-increaser", designed to utilize, for power purposes, freshet water otherwise going to waste over the dam, in the case of water-power plants, on rivers having a suitable régime, when situated adjacent to, or not far distant from, the dam. Mr. Herschel.

As the *Harvard Engineering Journal* is rarely seen, the results of the experiments alluded to may properly be recorded in conjunction with those referred to in the paper.

The experiments were conducted accurately at the Holyoke, Mass., public testing flume, all the water experimented with being measured continuously with two Venturi water meters. The "operating water" was in some of the tests as much as 14 sec-ft.; the "water lifted" (entering the tube by induced current), 7 sec-ft.; the natural, or "operating head", up to 14.35 ft.; the "head gained by the use of the fall-increaser", up to 12.52 ft.; and the penstock, 16 in. in diameter. With this apparatus, a vacuum, measured by more than 26 ft. of water column, could be produced and steadily held suspended; its efficiency, reckoning for water lifted a certain height, by water falling a certain height, as a

* This discussion (of the paper by F. zur Nedden, Esq., published in August, 1915, *Proceedings*, and presented at the meeting of October 20th, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

Mr. Hersche. maximum found, was 30.4%, which will be recognized, by those familiar with induction apparatus, as a high efficiency for appliances of that class.

The use of the "propeller sheet", noted in the paper, and apparently first used by Andres, in 1909, would, in the writer's opinion, materially increase this efficiency.

In effect, the apparatus tested was a form of injector; but, though the ordinary form of induction apparatus called an injector, or ejector, was tried, the writer found that it did not give encouraging results, compared with those produced by the form of apparatus herein described.

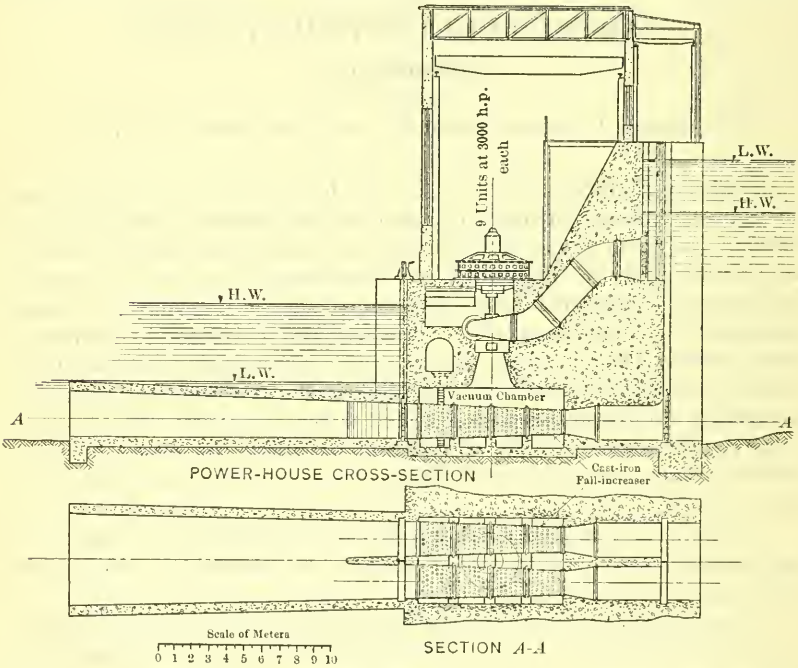

FIG. 27.

The casting, which serves as a mixing chamber for the "operating water" and the "water lifted", is not exactly a parallel case to an ordinary diffuser, because more water flows out of the down-stream end, than comes in at the up-stream end, in the proportion somewhat as 3 is to 2; but, for the purpose named, the best apex angle of the cone the frustum of which was used, was found to be 3° 30′. Its extension down stream had a cone angle of 5 degrees.

Fig. 27, which will explain itself, following the foregoing data, shows the application of the apparatus to the case of a water-power plant, not yet built, designed for the City of Geneva, Switzerland.

Carl George de Laval,* Esq. (by letter).—The author has brought before the Profession one of the most important subjects that interest designers of hydraulic machinery such as water turbines and centrifugal and turbine pumps. The paper brings out details which must be considered carefully in the design and selection of such machinery, and have only been known to a few within comparatively recent years. It is safe to say that a great many hydraulic machines have been designed by the purest guesswork, but, owing to the call for increased efficiencies and economy in operation in all the applications to various plants, the problems have received careful and intelligent study, causing many improvements, with more to come, when such papers as this bring them before the Profession for further thought. Mr. de Laval.

It is admitted that improvements are hampered by the scarcity of published data from experiments, and, therefore, such a collection of data as the author presents will give valuable information for the use of manufacturers.

In the evolution of centrifugal and turbine pumps no factor has played a greater part in obtaining increased efficiency than the study of the laws of the surface friction of fluids. This subject has been under investigation by the most eminent engineers for more than 100 years, and to-day the laws governing the velocity of flow for a particular fluid are tolerably well known. Experiments establishing a general relation applicable to all fluids and conditions of flow are not known, however, though it is known that such relations must exist.

A knowledge of the relations which hold between widely differing viscosities and densities, with reference to main flow, and also to secondary or induced currents, will be of great value. The author believes that such secondary currents exist or are co-existent at all flows, if the writer interprets the paper correctly. It is believed, and has been shown by experiments, that, at low velocities, there is no eddying; at higher velocities, however, these currents are co-existent with the main flow. There appears to be no doubt that Reynolds showed that the motion was in stream-lines, or lamellar in character at these low flows, and eddying or sinuous at the higher velocities, and that this change took place when the critical velocity was reached, which value is inversely proportional to the diameter of the pipe and directly proportional to the kinematical viscosity.

Induced Counter-Current Diffusors.—As the author makes special reference to turbine pumps, the conclusions drawn appear to compare well with results obtained by the writer in connection with similar theories put into practice, all of which point to the fact that the losses are due to the primary and secondary currents.

One of the most efficient diffusors designed to overcome these losses is that shown by Fig. 28. It consists of eight diffusor nozzles, starting

* Gen. Mgr., Worthington Pump Co., Ltd., Harrison, N. J.

Mr. de Laval.

with a rectangular section near the impeller periphery and then being shaped into a cone with circular cross-section, enlarging at the bend for the stream lines to follow the ideal fluid path, and reducing again to a circular section for the next impeller supply. This bend is circular, and will show a difference of pressure between the outer and inner points, due to the centrifugal force of the water flowing through them. This difference can be ascertained. The result is a vortex set up at the inner surface, which can be eliminated when the water is swept clean, due to secondary currents which also set up a rotary motion while passing through the diffusors.

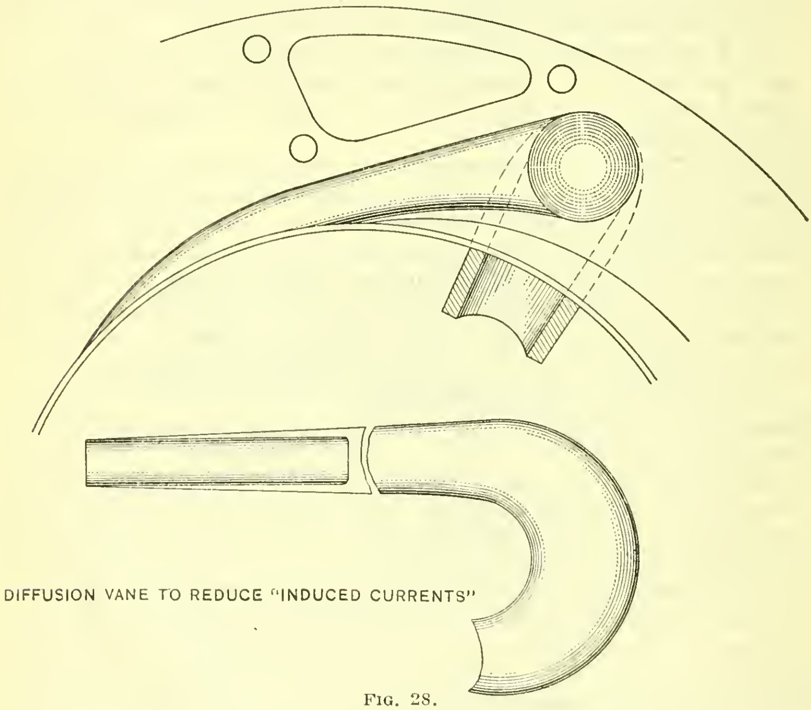

FIG. 28.

The discharge from these eight diffusors is exactly the same as that from eight nozzles discharging parallel to the axis of a turbine pump. This arrangement appears to bear out the author's theory, and was applied by the writer on some important installations of turbine pumps with 250 ft. per stage, and gave a remarkable efficiency of 86 per cent. There was every indication, after the experiments were made with diffusors of this shape, that they gave a diffusion efficiency as high as 99.5%, and fully substantiated the proofs of Andres' experiments.

Transverse Induced Currents.—In considering this subject the writer believes that there are two critical velocities which must be taken into account: one is the velocity due to the fluid entering with a high state of turbulence, passing from an eddying to a stream-line motion; the other is that in which an undisturbed fluid enters with a stream-line motion which exists until the critical velocity is reached Mr. de Laval.

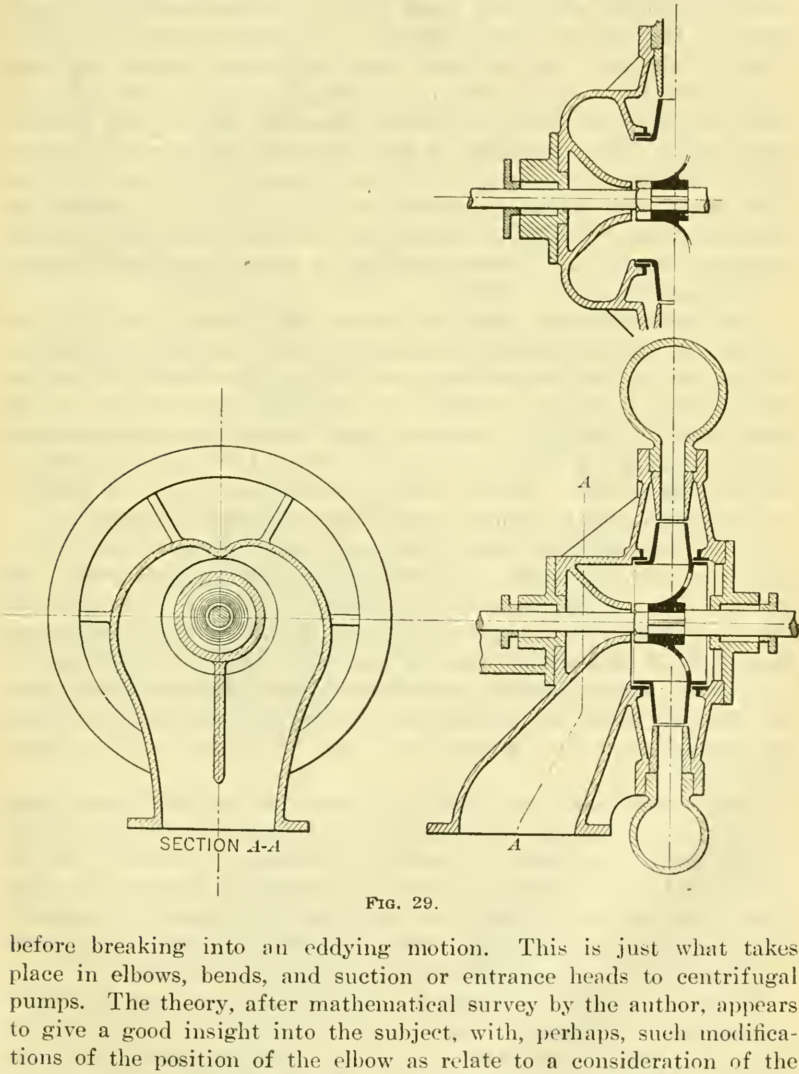

FIG. 29.

before breaking into an eddying motion. This is just what takes place in elbows, bends, and suction or entrance heads to centrifugal pumps. The theory, after mathematical survey by the author, appears to give a good insight into the subject, with, perhaps, such modifications of the position of the elbow as relate to a consideration of the rotation of the impeller which is to receive this water finally.

Mr. de Laval.

Fig. 29 shows a design of the suction entrance of a pump having one turn or elbow for the water. This embodies the theory of gradually increasing the velocity in order to produce a vortex at the impeller hub. It is to be noted that the upper section is divided like a heart, and stream lines are directed toward a cone at the impeller entrance.

Fig. 30 shows a design of an entrance intended to obviate these induced currents, where the water has to enter horizontally at right angles to the rotation, and is properly fed or directed to the runner. Owing to the fact that the water receives a rotary motion, and that in this case there are a great many induced currents, the inflow of water to the runner would be greatly impaired and probably reduced considerably. This was shown by experiments similar to von Cordier's tests indicating by pressures at various points the various areas and places where these induced currents took place; they also revealed the fact that it was necessary to prevent the whirling of the particles in an irregular manner and direct them into a well-defined path forming two or more streams or currents.

The results showed that this theory was correct, and that, by making the suction chamber heart-shaped, shutting off the supply to the runners nearest the entrance, and dividing the stream into two or more nozzles, producing a volute chamber, the rotary motion of the water, based on the law of rotation, swept the entrance channels clean, thus preventing a critical velocity which would have started or broken into eddying motion. The final results showed cases where the capacity was increased from 40 to 50% on otherwise the same construction.

In the conversion of kinetic into pressure energy, such as is to be considered in the design of hydraulic piping and machines, the passages have to be increased or decreased suddenly to reduce the velocity of flow, and in converting a part of this into pressure energy many difficult problems are met, the laws of which are but little known. If, however, experiments are undertaken to establish these points, data are obtained which are suitable for all practical purposes, and these few results, given in a general way, may aid others to derive some information.

The author points out that if designers of hydraulic machinery would train themselves to produce correct forms, conclusions would be diametrically opposite to practical experience. This may not be fully understood, as theory cannot be correct if it does not fully agree with practice. It might be said that the hypothesis or assumption of designs does not agree with facts, because a great many facts in the design of hydraulic machinery are borne out by theory.

In Fig. 25, the author shows a series of auxiliary vanes or profiles to impellers which must be based on the theory that water enters the inner periphery of the wheel in a radial direction across the entire width of the wheel. It appears that such construction would increase the induced current and aggravate the situation rather than relieve it.

Mr.
de Laval.

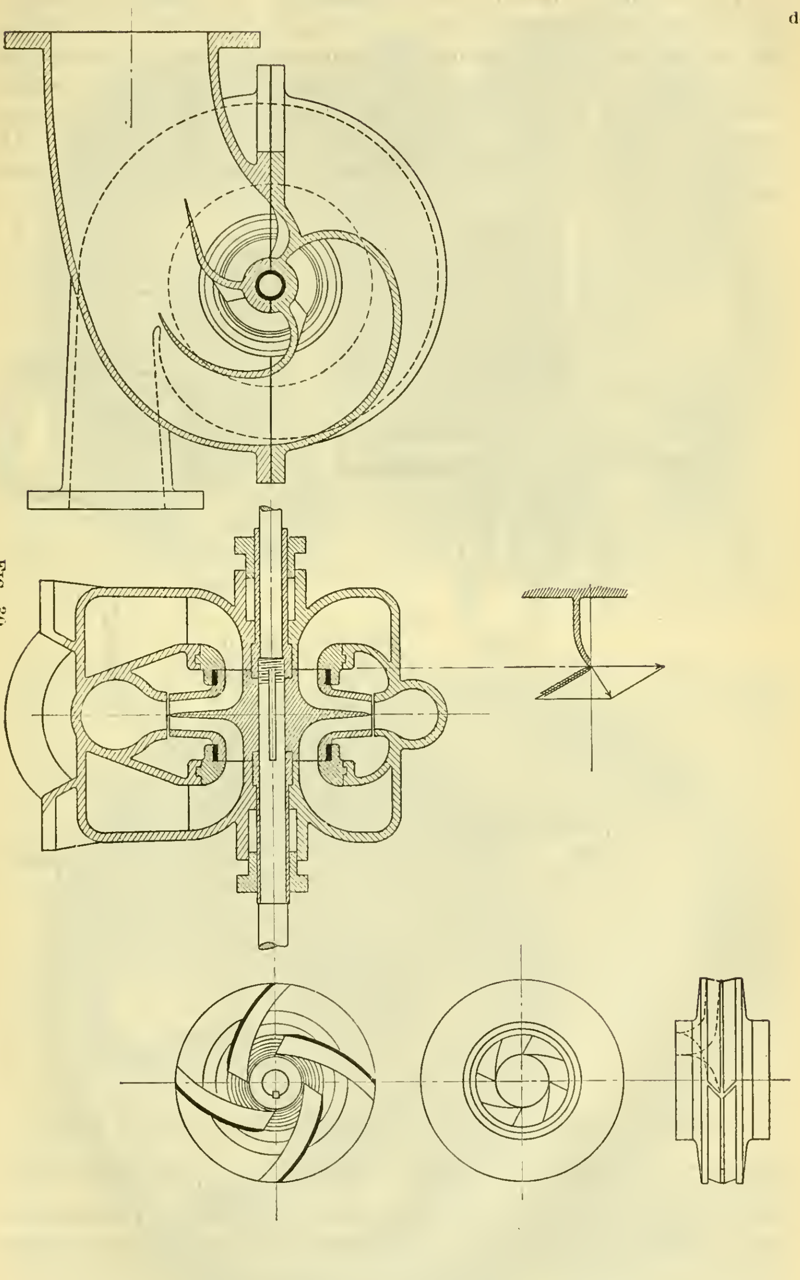

FIG. 30.

Mr. de Laval.

Fig. 31 shows a design of an impeller with an entrance elbow containing the very opposite of the author's auxiliary profile, which design has shown that it will reduce the induced currents and prevent the stream from breaking into eddying or sinuous motions. It also prevents the water from changing into abrupt directions which would produce churning action, reducing the capacity and efficiency. It gives the impeller or wheel the full energy of the water in the proper action of the fluid.

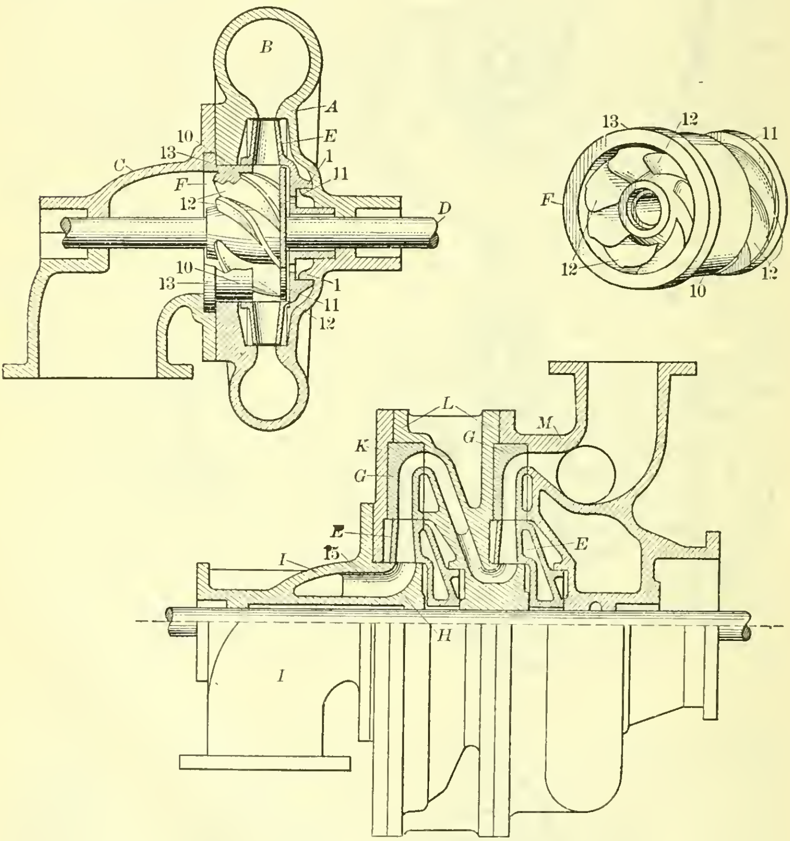

FIG. 31.

There is a reference by the author to difficulties of design in impellers on low-lift pumps when the ratio of $\frac{D_2}{D_1}$ is small, and he points out the difficulties with small ratio of impellers. Impellers for such conditions—high speed and low lift—are being made with equal

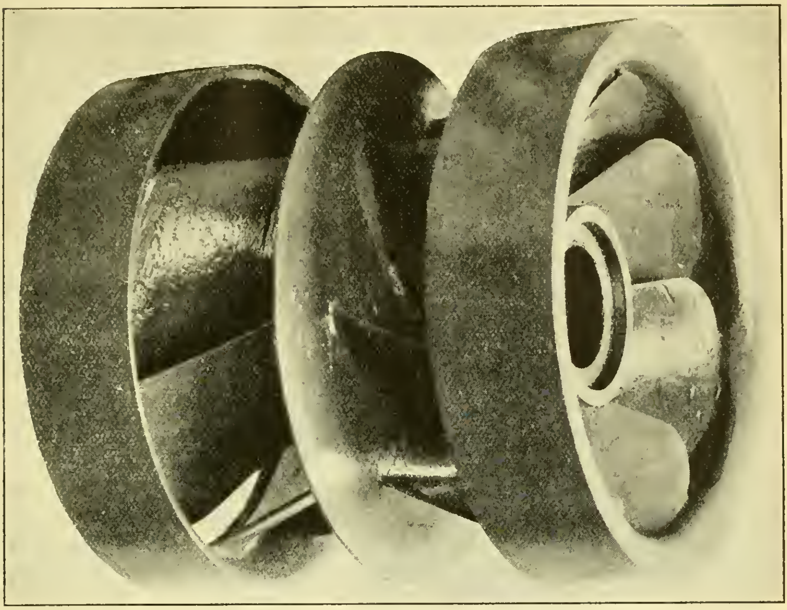

FIG. 32.

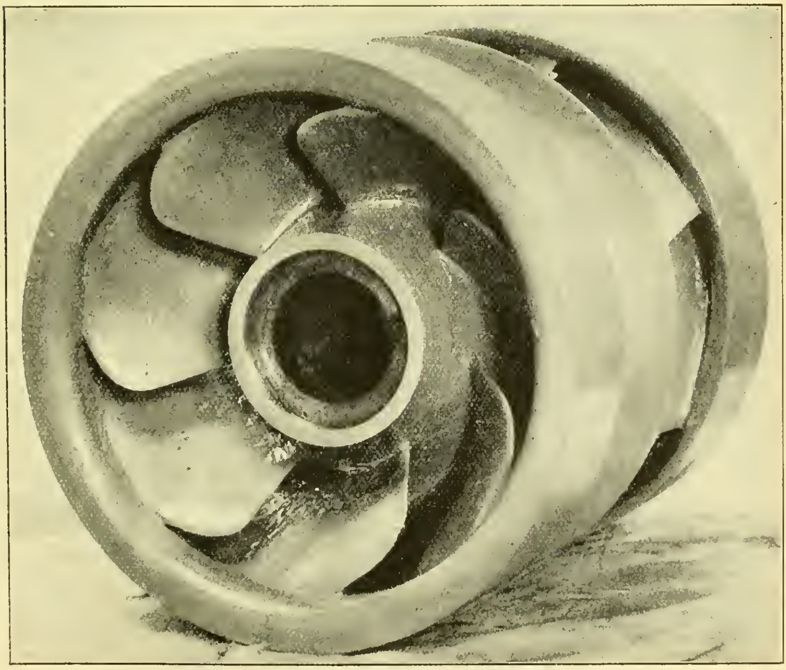

FIG. 33.

efficiency to high-lift pumps by adopting designs such as those shown by Figs. 32 and 33, which indicate clearly the developed vanes extending through the impeller entrance to the other end of the vanes developed on the cone to the outer circumference which, in some cases, can be made smaller than the entrance diameter of the hub of the impeller. Such impellers give a remarkably high efficiency for low heads and high speed. The method of layout for such vanes is shown by Fig. 34. Mr. de Laval.

Another method for inducing the flow to the impellers by guide nozzles is shown in Fig. 35. These references are given to show that there are other ways of obviating the defects referred to by the author, without resorting to expensive auxiliary partitions in the impellers.

In the author's reference to diffusor vanes, it is stated that the water refuses to be guided. The writer's experiments show that water can and must be guided, but, at the same time, it must be done with due regard to the main motion of the stream lines and the secondary motion of the induced currents, introducing the necessary twist or corkscrew action with a nozzle discharge, which, so to speak, clears the passages. The writer believes that the unrestrained particles of water from the periphery of a rotating impeller will leave in a tangential direction at a tangential velocity. As the quantity is increased, the direction and velocity are modified by the direction and velocity of the radial flow.

The proportions of conical nozzles are known, in which velocity is converted into pressure with the least loss of head, and it is generally accepted that a conical pipe with a proper indirection to the axis is the best. As it is possible to surround the periphery with a number of such conical nozzles arranged so that their axes are in the direction of a particle of water leaving the impeller and of such an area at the throat as to suit the quantity and velocity of the water delivered, it would seem as if the highest efficiency of conversion would be reached.

The influence of runner curvature on the discharge velocity in centrifugal pumps is illustrated by the diagram, Fig. 36. This represents a 54-in. double-suction drainage pump, taken about 12 ft. from the pump nozzle. Of peculiar interest will be the drop in velocity in the center of the pipe when compared with the typical velocity diagram, Fig. 2, and with Fig. 25. Similar results were obtained with a number of different pumps. The disturbance is especially noticeable in high-speed pumps, due to the small curvature of the impeller as well as the small radius of the casing.

In Fig. 25, the author shows the velocity diagram of a high-speed impeller with radial components of respective pressures. Experience has shown, however, that the direction of flow converges toward the

Mr. de Laval.

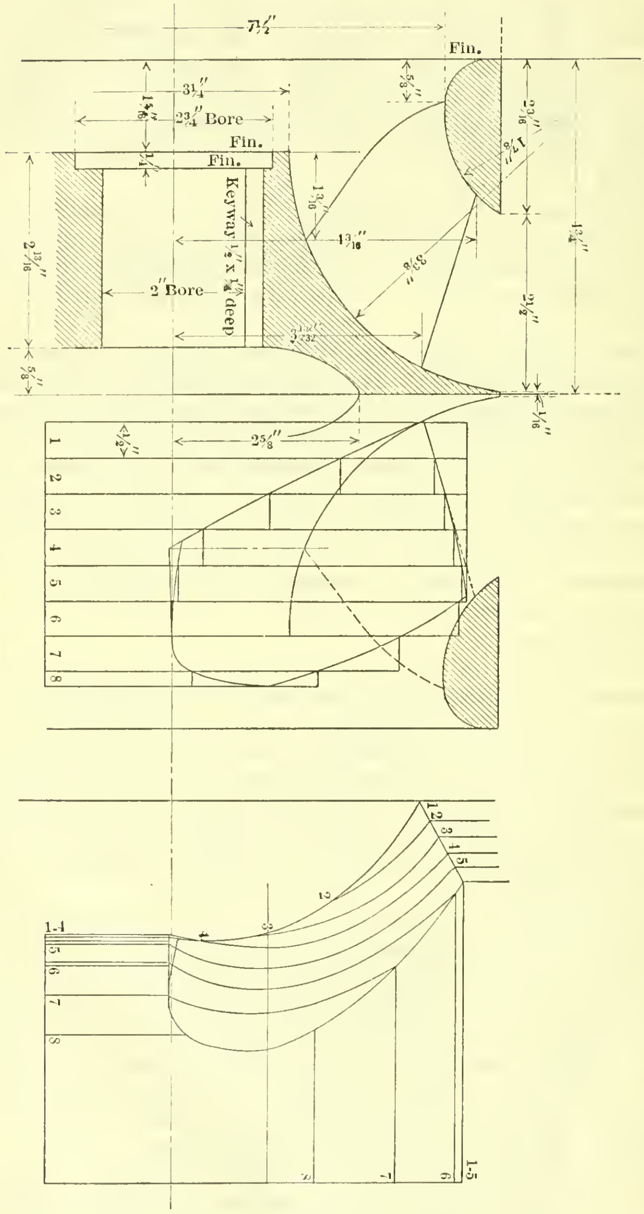

FIG. 34.

Mr. de Laval.

center, even if the ends of the curvature point radially. (See Fig. 37.) This fact must be kept in mind when designing the volute casing throat, and will be found of extreme influence in improving the efficiency.

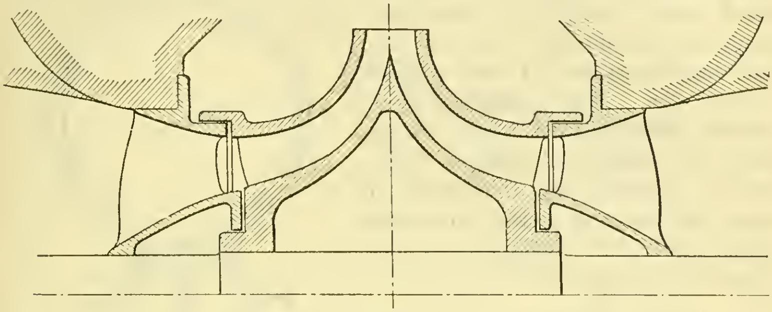

FIG. 35.

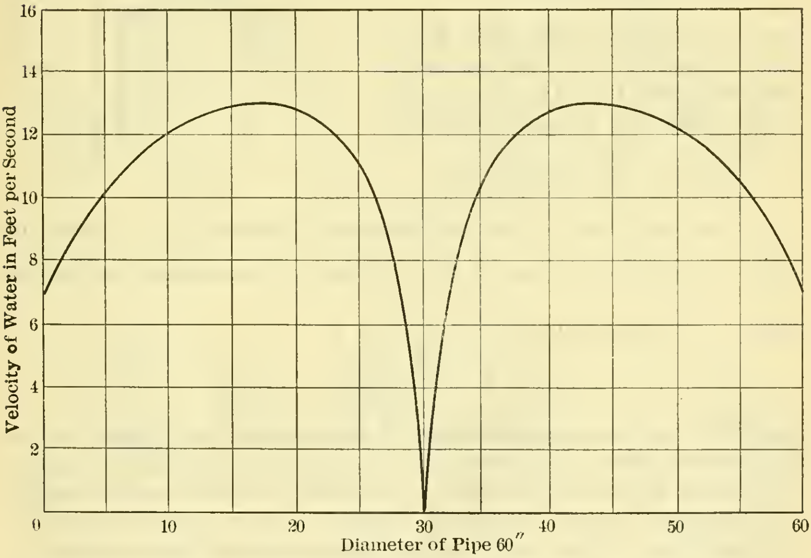

FIG. 36.

Impellers with auxiliary profile, as suggested on page 1399,* were tried by the writer in 1910, but without showing any improvement over the ordinary type, and have been given up. Subsequent experiments have shown that the degree of curvature has very little to do with the efficiency of the pump, provided proper attention is paid to the shape of the inlet. The common suction elbow will not do. Fig. 38 shows a series of experiments made on a high-speed pump

* *Proceedings*, Am. Soc. C. E., for August, 1915.

Mr. de Laval.

by simply changing the contours of the suction approach and the suction velocity. If it is considered that the efficiency of the pump was raised from 40 to 71%, without any change in the impeller or in the casing, the importance of this will be readily seen. The influence of a properly directed inlet is not so apparent on slow-speed pumps. This can be explained by the speed at which the water is picked up by the vanes. Take, for instance, an impeller with a ratio of

$$\frac{\text{outside diameter}}{\text{suction inlet diameter}} = 2,$$

then the velocity of the inlet, U_s, is equal to one-half the velocity at the outer diameter, U_2. Any shock losses at the inlet will be proportional to

$$\frac{U_s^2}{2\,g}, \text{ or } \frac{1}{8}\,\frac{U_2^2}{g}\; L = C\,\frac{U_2^2}{8\,g}.$$

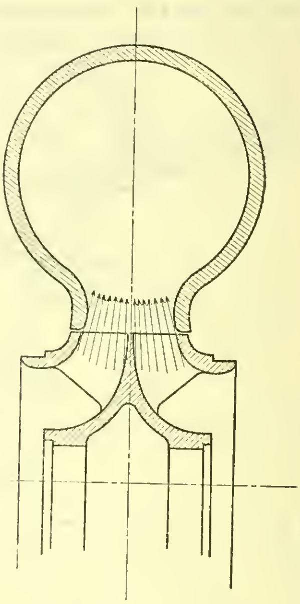

Fig. 37.

Let us take now an extreme high-speed impeller with a ratio of $\frac{D\,a}{D\,s} = 1.15$, then the loss at the inlet can be ascertained in a similar manner to the proportion, $\frac{1}{2.66} \times \frac{U_2^2}{g}$, or

$$L_1 = C\,\frac{U_2^2}{2.66\,g}.$$

Now U_s^2, or the peripheral speed of the impeller, is proportional to the pumping head. Any losses at the inlet due to shock will be in the ratio of 8 to 2.66, or 300% higher in the case of the high-speed impeller.

High-speed pumps, therefore, demand a most careful study of these problems, or they will be very disappointing in their results.

Elbows in the suction line close to the pump may have a great influence on the characteristics of the latter. Fig. 39 shows a 14-in. pump. Arrangement "A" was made with a standard long-sweep elbow, 3 ft. from the pump nozzle, with a pump efficiency of 65 per cent. Arrangement "B" was made subsequently, by which the efficiency was raised to 75% and the head curve was improved likewise. Another bad effect of elbows in a suction line of double-suction pumps will be noticed in the considerable end thrust.

A great deal of credit is due the author for this paper, as it gives such valuable information on an important subject about which little is known.

Mr. de Laval.

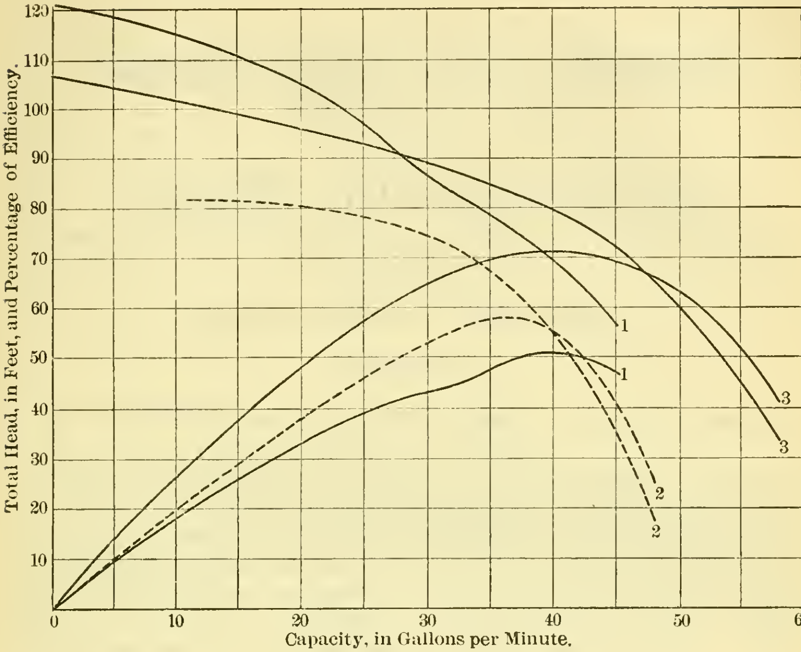

FIG. 38.

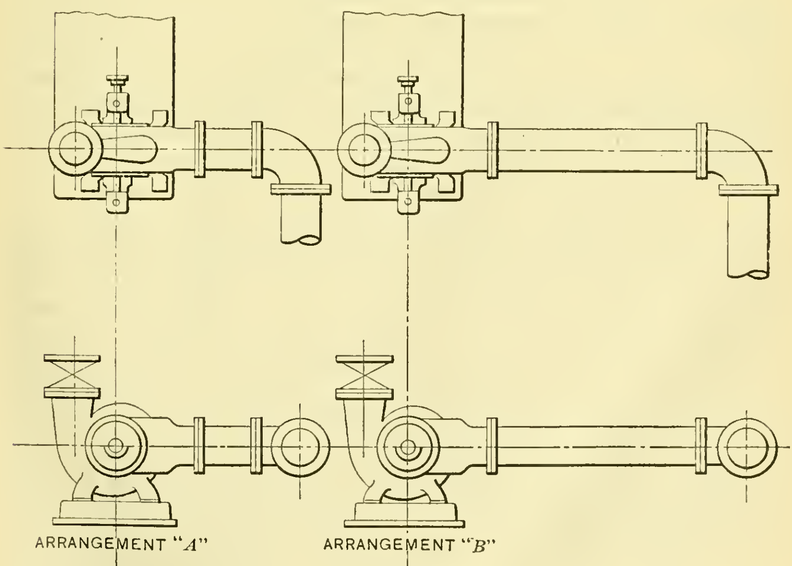

FIG. 39.

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PAPERS AND DISCUSSIONS

PEARL HARBOR DRY DOCK

Discussion.*

By Messrs. Leonard M. Cox, W. H. Pretty, and J. R. Baterden.

Leonard M. Cox,† M. Am. Soc. C. E. (by letter).—This paper covers so thoroughly the history of the Pearl Harbor Dry Dock construction to date, as well as the difficulties encountered in connection therewith, that little remains to be said in the way of discussion, beyond congratulating the author on the novel design finally evolved for surmounting those difficulties, and on the very valuable contribution he has made to the literature of maritime engineering. Mr. Cox.

As two plans were mentioned in the paper as having been suggested by the writer, it may be of some slight interest to explain these plans, or schemes, for carrying out the work, in a little more detail, and to describe briefly the conditions which led to their submission. The first hardly deserves the title of "plan", as it never progressed beyond the sketch stage, and for the further reason that conditions at that time (June, 1911) were not such as to warrant serious consideration of a change either in design or in method of construction. It consisted in the building, on floating submersible platforms, of a number of reinforced concrete floating docks, of convenient length and of a width corresponding to that of the completed dry dock, the depth of pontoons and width of side towers to be such as would allow ample room for placing the lining, when in position, as a part of the completed structure. The floating docks were to be towed to their destined locations and sunk, by the admission of water, as adjoining sections of the dry dock. When landed on the previously prepared bottom, the interior chambers were to be filled with concrete, the joints between adjoining pontoons, or floating docks, were to be made with tremie-placed concrete, and the dry dock floor and side-walls com-

* Discussion of the paper by H. R. Stanford, M. Am. Soc. C. E., continued from October, 1915, *Proceedings*.

† New York City.

Mr. Cox. pleted to design lines after unwatering. It is unnecessary to state that this plan would have involved great expense, a nice preparation of the bottom under difficulties, and the expenditure of much care and skill in the placing of the floating dock units.

In order to explain the second scheme, it is necessary to review briefly the conditions as known at that time. As stated by the author, the first attempt to unwater occurred about May 1st, 1911. This is fully described in the paper, and it will be recalled that the water level was reduced 16 ft. before evidence of bottom upheaval was observed, and, during attempts to unwater beyond this point, the bottom of the excavation was not ruptured. These circumstances led to the belief that if that particular section was typical of the entire area of

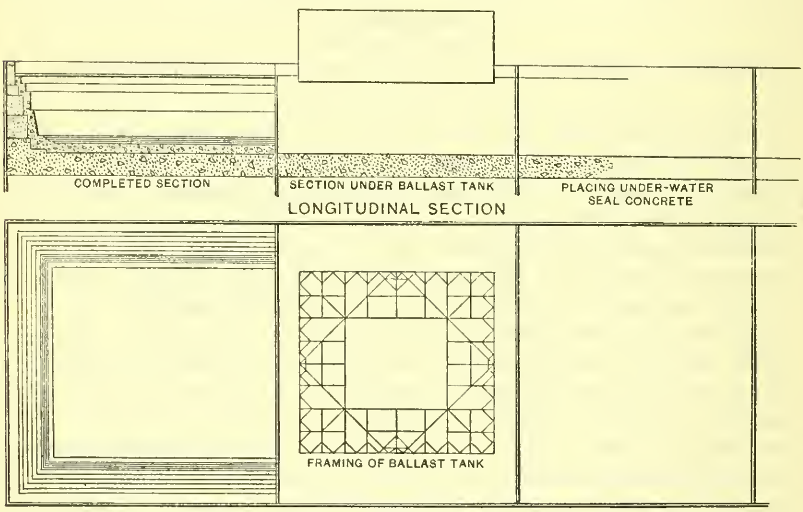

Fig. 22.

excavation floor, there existed below the dock prism a more or less impervious layer of such a thickness that its weight was equivalent to that of 16 ft. of water, a conclusion which was concurred in by the contractor's engineer.

As both the contractor's engineer and the Government engineer believed, from their experience on the work, that it was possible to drive piles 35 ft. into the underlying material, an agreement was reached by which additional compensation was provided for placing piles throughout the bottom, these piles to pierce the impervious layer on which the dock was to rest and pin into the material beneath. The notched heads of the piles were to project 4 ft. into the floor concrete, and the contractor was to be allowed to place not more than 8 ft. of sealing concrete by the under-water method. It was believed

that the anchorage value of the length of piling which would penetrate the material underlying the impervious layer, together with the weight of under-water placed concrete and that of the impervious layer itself, would safely counterbalance the hydrostatic pressure which would exist when the excavation was unwatered. As a matter of fact, however, it was found to be impracticable, if not impossible, to drive piles of this length. Those driven in Section II, averaged a penetration of 21½ ft., with minimum penetrations as low as 7 ft., giving no anchorage value below the impervious layer. In this same section the concrete seal averaged 6½ ft. with a minimum of 3 ft. Of the entire section area, 55% contained piles which barely penetrated the assumed 19 ft. of impermeable material. Mr. Cox.

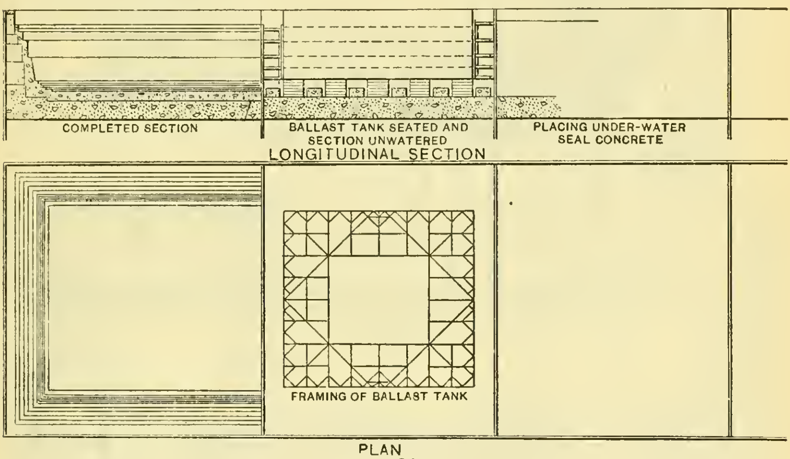

FIG. 23.

The writer was called in consultation in connection with the renewed studies and investigations following the second failure to dewater in February, 1913. Assuming that the failure was due to a lack of anchorage which prevented the unwatering of the partly completed sections necessary for the completion of the dock in the dry, and agreeing fully with the opinions of Mr. Noble and Mr. Harris in regard to the impracticability of using stone ballast, he devised a method of construction involving the use of water ballast. This method, he believed, and still believes, would have insured the completion of the dock by the original design, modified only by comparatively insignificant increases in the quantity of concrete involved and by the addition of certain reinforcing steel to compensate for the lost holding-down power due to short piles. He further believed that the adoption of such a plan would go far toward eliminating the risk of the original crib method, and would expedite completion.

Mr. Cox. In its details, the plan proposed no change in the contractor's bulk-headed sections, or in the preparation of his foundation by the driving of piles to the greatest penetrations possible to obtain. The heads of these piles, as called for in the original contract, were to be embedded

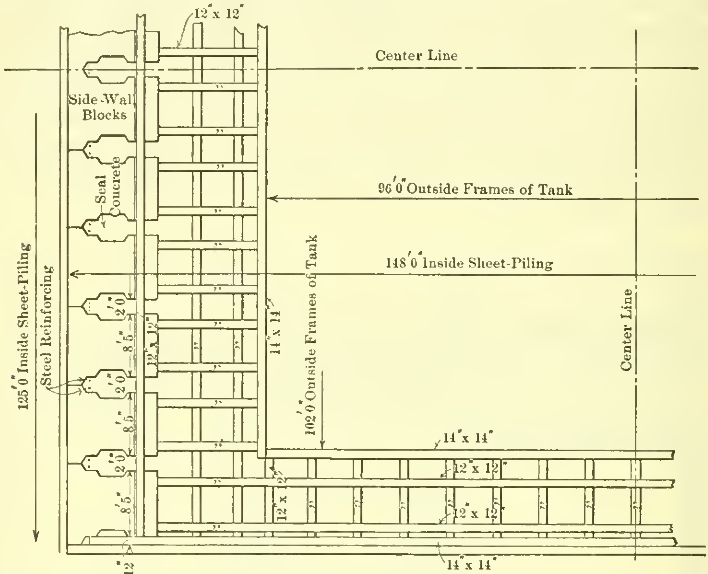

FIG. 24.

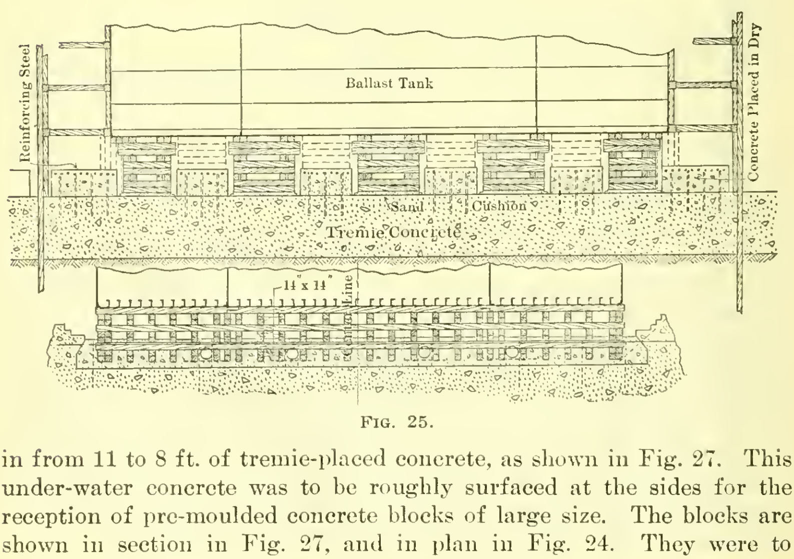

FIG. 25.

in from 11 to 8 ft. of tremie-placed concrete, as shown in Fig. 27. This under-water concrete was to be roughly surfaced at the sides for the reception of pre-moulded concrete blocks of large size. The blocks are shown in section in Fig. 27, and in plan in Fig. 24. They were to

be placed by the 150-ton floating crane already provided for the Pearl Harbor Station. After the piles, tremie concrete, and side-blocks were in place, a steel ballast tank was to be floated into position and sunk by the admission of water to a bearing on timber or metal keel-blocks arranged by divers across the dock section. Then, by pumping, the level of the water in the ballast tank was to be raised some 8 or 10 ft. above the level of the water in the excavation. With the tank resting on the underwater-placed floor, the next step in the operation Mr. Cox.

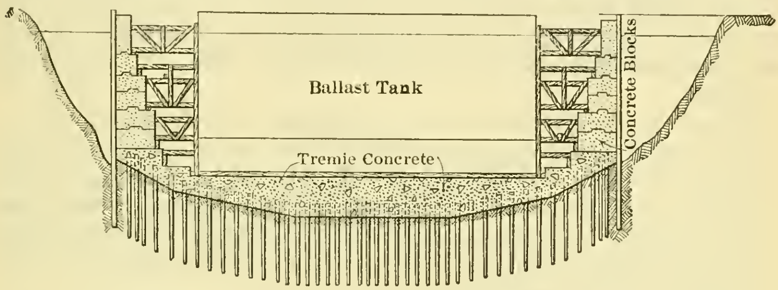

FIG. 26.

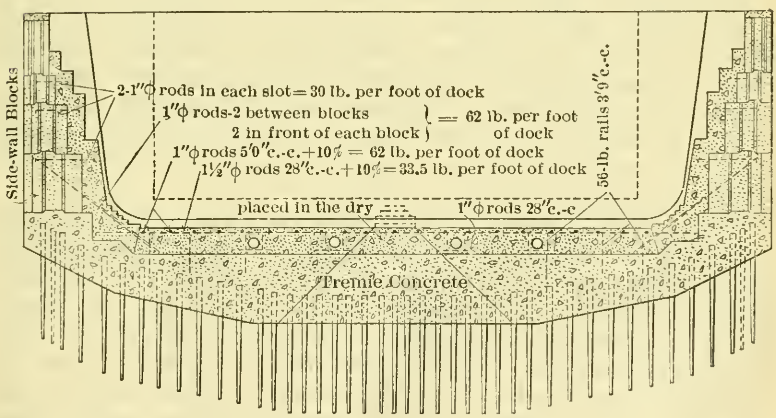

FIG. 27.

involved the unwatering of the space between the section bulkheads and the tank, shoring from the bulkheads or side-wall blocks to the tank sides as the pumping progressed. On the completion of pumping, the floor strips between the keel-block rows were to be completed in the dry, as shown in Fig. 25. The keel-blocks would then be shifted to these completed strips or ribs, and the space thus vacated completed in the same manner. The lower tier of shores between the ballast tank and the side-wall blocks was then to be removed, and the lining of the side-walls was to be placed, working from bottom to top. With

Mr. Cox. the entire section completed as described, water was to be admitted; then the ballast tank was to be pumped to light draft and floated over a shallow cut in the athwartship sheet-piling bulkhead to the next section, where the operations were to be repeated. The entire operation is illustrated in Figs. 22 to 27, from which it will be seen that ample provision was made for bonding the tremie concrete and that placed in the dry; also, that horizontal steel was introduced at the top of the floor-slab. The side-wall blocks were provided with female keying spaces on the sides, as well as on the tops and bottoms, the side spaces permitting the emplacement of steel for side-wall strains, and the horizontal spaces facilitating the free flow of thin sealing concrete.

The plan was submitted in the belief that such design changes as were involved would fall under the head of minor changes, as provided by the terms of the contract. It involved no essential change in method of construction, for, like the contract design, it was based on the principle of compensating for the weight of water and earth removed, either by anchorage value or dead weight, during the entire course of building operations.

Although, as stated previously, the writer still believes this plan to be in every way practicable, relatively economical, and relatively safe, he is of the opinion that, aside from legal obstacles concerning which he is not qualified to judge, the adoption of the more ingenious scheme proposed by Admiral Stanford was fully justified by the advantage of securing a stronger structure than was originally contemplated and of eliminating any doubt which may still exist as to the reliability of under-water-placed concrete.

To those at all familiar with the marvelous growth in sizes of ships during the last decade, the three changes in design dimensions described in the paper will not appear to be strange. In 1909, no battleship approached 581 ft. in length, nor 100 ft. in beam. In 1910, a battleship cruiser 625 ft. long had become a reality in foreign navies, and the Panama Canal locks had been fixed in width at 110 ft. In 1914, 700-ft. warships were no longer considered impossible, and provision for docking facilities for the modern passenger liners in time of war had become a real necessity. At the date of writing this discussion, battle cruisers 800 ft. long and of 100 ft. beam are seriously considered. The Department is to be congratulated on its broad policy of progress and on its determination to keep pace with the advances in every branch of science or art relating to its work.

Mr. Pretty. W. H. Pretty,* Esq. (by letter).—This paper, on perusal, conveys the impression of a vast piece of engineering research, from which the engineering world in general may read and learn. The plain

* Peterborough, Ont., Canada.

statement of difficulties met and to be overcome almost disarms criticism, and the determination to carry out the original scheme of building a graving dock in the face of such difficulties, was heroic. The writer's first impression, after reading the paper, was that the position is ideal for a floating dock in preference to a graving dock. The author, however, has already dealt definitely with this question. and no further reference is necessary. The work in hand is the building of a graving dock under extremely adverse conditions, and the thoroughness of investigations, recorded by the author, and the methods determined on to overcome the difficulties, excite admiration and respect for those responsible for such work, carried out so far from their base. Mr. Pretty.

The writer would suggest that a lay-out plan of the dock, giving local contour lines above and below the water line, referred to "mean low water" (to which it is presumed the depth of 35 ft. over the sill of the graving dock is referred) would add greatly to the interest of the paper. It would also be of value to have on record the degree of curve decided on for the safe passage of modern battleships, and the corresponding speed in knots; and, further, if any portions of the channel leading to and from the graving dock are provided to act as turning basins for large craft. In view of the important position in the Pacific of Pearl Harbor and Honolulu as "safe harbors", the writer would ask whether any consideration has been given to the use of the graving dock by ocean liners in general, which tend to outgrow in dimensions, tonnage, and draft the most powerful modern warships.

The results of investigations relative to the value of under-water-placed concrete by "tremies" are interesting and valuable, and the practical abolition of this method of placing concrete, except for auxiliary purposes, in the final plans for the graving dock, is sufficient and eloquent confirmation of the ever-present element of hazard in such large and important undertakings. Expensive "forming" is required in any case, and thoroughly water-tight, for reasonably satisfactory work, especially where there is stream flow past the coffer-dam. It is, however, a valuable aid for auxiliary purposes, and where the concrete thus placed is extraneous to the structure to be built, such as acting as a bottom seal for coffer-dams or to form a better bedding than the existing bottom provides. In the writer's opinion, a structural steel coffer-dam, with steel-plate shell, the whole of the lower part of which could be left permanently in the structure, would amply repay the first cost, in saving of labor, removing hazard, and in future security, and is an investment of the "safety first" order. The use of the "coffer-dam boat", described by the author, is unique, and suggests many possibilities for analogous use in building other structures and substructures, and the water tanks possess many advantages

Mr. Pretty. for loading and unloading coffer-dams to meet the exigency of the moment.

The experiments on various concrete mixtures are full of interest, and seem to confirm the superiority of clean silicious sand in making "cement mortars". The writer's experience confirms this, and he is of the opinion that some form of surface chemical action takes place between the colloid silica in solution, in the newly-mixed, moist, cement mortar, and the clean quartz grains, probably in the nature of a process of segregation around these grains, which, when in close proximity, become cemented into a solid mass, improving with age; and when widely separated, or intermingled with other materials not amenable to this action, producing a weak concrete, deteriorating with age. We have instances of analogous segregation in the flints formed in chalk beds and calcareous sandstones; the kernels of the former are frequently pebbles, and occasionally have metallic nuclei of fossil origin. The writer has formed the impression from the latter portion of the paper that the use of silicious sand in making the concrete has been abandoned in the final plans for the dock, and that a form of so-called "puzzolan" mixture has been decided on. It would be interesting to know if the value of the latter for depositing in sea water is as great as generally reputed. Cements are often blamed when the mixing and "forming" are at fault, but there is no doubt, however, that each case must be dealt with on its own merits and from the standpoint of the nature of the water in which the concrete is to be deposited—be it salt, or brackish, or fresh water—and the paper certainly conveys the idea that this has been done in the work described.

It may not be without interest to mention here that the placing of under-water concrete has been reported to have been carried out with special dump scows, the concrete being placed in large, specially made, sail-cloth sacks, or bags, firmly sealed up before dumping on the site. The method has its possibilities, and doubtless its limits, when dealing with large undertakings.

The use of vertical tie-rods in the concrete near the outer faces of the walls is commendable. The writer does not recollect seeing any reference in the paper to the size of individual bars to be used for this vertical reinforcement, but may have overlooked this. Generally speaking, he is of the opinion that vertical ties near the faces of walls subject to hydraulic pressure should be individually of large sectional area with substantial anchorages, in preference to a large number of the usual type of small-section deformed bars, the latter being used merely as auxiliary reinforcement. The usual practice of artificially raising the elastic limit to an "excessive degree" by twisting and other deforming processes should be condemned.

Mr. Pretty.

The final plans, with all their completeness for a successful issue of the work, make one hesitate to make any comment save of appreciation for the thoroughness and ingenuity, combined with unique methods, which they show. The writer, however, would like to ask the author if the possibility of building the side-walls of the graving dock independently, received consideration, placing the dock floor later. This would seem to be, at first sight, a likely solution to the problem, the side-walls being built in alternate sections with coffer-dams, such as indicated by the writer in a previous paragraph, using an adaptation of the detachable coffer-dam boat of the author for the removable upper works, and leaving the lower part of the coffer-dams permanently buried in the concrete, the intermediate sections of the walls being built by the use of overlapping coffer-dams, checks being left in the base of the inner face of each side-wall into which the dock floor would lock. The dock floor would be laid in sections of convenient size, similarly, the ends of the coffer-dams in this case sealing with the inner faces of the side-walls of the dock. The tremie concrete in all cases acting as a seal only, would enable the actual true concrete of the structure to be placed in the dry, as indicated by Fig. 28.

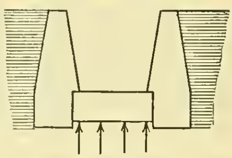

FIG. 28.

The adoption of main centrifugal pump units with vertical shafts, each direct-connected to a three-phase motor, forms an acceptable main pumping plant for a graving dock, where a shore or floating power station is available, and, as similar pumping installations have been already installed elsewhere, it would be interesting to know if the 2 200-volt motors have proved all that can be desired under the trying conditions of pumping against varying heads, stopping and starting, to meet shoring requirements, etc., in the operation of a graving dock. At first sight, the writer would prefer 550-volt motors, reducing the size of the units, if necessary, and using more of them. Under the latter conditions, it should be possible, with three-phase motors, having suitable resistances in the rotor windings, connected up to the necessary controllers, etc., to have sufficient speed variation available to enable the motors to give out full brake horse power (approximately) throughout the whole period of pumping. The efficiency, from an electrical standpoint, might not be ideal, but it would have many practical advantages. It should not be forgotten, when considering the efficiency from a theoretical standpoint, that the pumps and motors may be idle for intervals extending into periods of weeks and months.

The reference to Artesian pressure and the tests carried out, make interesting reading, bringing to mind our school days and the teacher's

Mr. Pretty. demonstration of the relative density of salt water and fresh water, in its application to engineering problems of to-day. Coming to the question of earth resistance vertically to loads placed on its surface, generally speaking, it may be taken that the average earth resistance over a large surface per unit area is greater than that obtained by tests made on relatively small isolated areas on the same site. In driving piles in "mud bottoms", it should not be forgotten that in such bottoms the hydrostatic uplift must be referred to the "lower ends of the piles themselves". A case came under the writer's observation some years ago where a large store shed (the floor of which was a concrete cap to a pile foundation in a mud bottom) rose 8 in. or more less than 2 years after the completion of the building. The site was on the marshes at the mouth of a river where the mud deposit was wet and of great depth. The rising was irregular, and the brick walls cracked in several places.

The idea of using a floating dock for the section bases speaks for itself, and deserves none the less credit on this account. A little more information about the timber floating dock to be used would be acceptable; for instance, are each of the five pontoons subdivided into chambers, each under independent control during docking operations? Again, are the side-walls also to act as independent pontoons for assisting in controlling docking operations? The writer would recommend that the ballast, which is to be placed on brackets on the inner faces near the top of the side-walls, be enclosed and under control, if such ballast is really necessary. There is nothing worse than moving ballast in the event of accidental heeling over from any cause whatever, whether the ballast is solid or fluid. To the writer it seems a pity that an appropriation was not secured for making this a small up-to-date floating dock of steel throughout, as it would prove a valuable auxiliary unit for use in Pearl Harbor and Honolulu after completing its work on the Pearl Harbor Graving Dock, for, in addition to its use as a floating dry dock for small craft, it would make a magnificent floating repair yard for ships in open water. It could also act as a floating power station and for salvage purposes, etc.

The use of structural-steel built-up frames, for embedding in the concrete of the bases, is a masterly solution of the problem of dealing with the dock floor, and forms a ready means of attaching stiffeners, etc., for the side-walls. The flat bottom adopted in the final plans for the graving dock is unquestionably the best for such a location and under such conditions as exist there. In the blasting operations carried out for the bottom bedding, is it not possible that the blasting charges may be too heavy? Smaller charges and more of them are better for foundations, as large charges disturb the bottom excavation very severely and very irregularly. The excess of weight of dock of

12 840 lb. per lin. ft. over hydrostatic pressure, referred to base of dock, does not appear to be very large, being apparently only about 2% excess per unit area of bottom of dock; this excess might be increased with advantage to stability. Mr. Pretty.

Concrete loaded with scrap structural-steel sections and punchings, etc., is occasionally useful, and particularly where a reduction in volume for a given weight is absolutely necessary, as occasionally happens in counterweights, etc. Blocks of such loaded concrete have been used by the writer, and, from actual measurements and weighing after manufacture and drying out, and where the concrete was apparently loaded to capacity with scrap metal as stated, the weight came out very approximately at 210 lb. per cu. ft. The use of such concrete in large quantities, however, would prove rather expensive.

Taking the condition of a heavier-than-water structure, it will be interesting to see the balance sheet of weights, and buoyancy, and the resultant "negative buoyancy" (resultant downward pressure) to the credit of the material of the dock, referred to the case where the dock is empty.

Considering, for the moment, the dock as a floating structure, with sealed ends, and "empty": let the volume of material of the dock $= V'$ and its average density $= \rho'$; the volume of the material of the dock below water line $= B$; and the volume of the material of the dock above water line $= C$.

$$\text{Then } V' = B + C \text{........................} (1)$$

Let the volume of the empty space inside the dock "below" the water line equal A, and the total volume of fluid displaced equal V and its density equal ρ.

$$\text{Then } V = A + B \text{........................} (2)$$

$$\text{and, } (A + B)\,\rho = (B + C)\,\rho' \text{.....................} (3)$$

$$\text{or, } A = V'\,\frac{\rho'}{\rho} - B = V'\left(\frac{\rho'}{\rho} - 1\right) + C \text{.........} (4)$$

If we take $\Delta\rho'$ as the allowance for resultant downward pressure, then

$$A = V'\left(\frac{\rho' - \Delta\,\rho'}{\rho}\right) - B = \left(\frac{V'\,\rho'}{\rho} - B\right) - \frac{V'\,\Delta\,\rho'}{\rho} \text{....} (5)$$

$$\text{or, } A = V'\left(\frac{\rho' - \Delta\,\rho'}{\rho} - 1\right) + C$$

$$= \left\{V'\left(\frac{\rho'}{\rho} - 1\right) + C\right\} - \frac{V'\,\Delta\,\rho'}{\rho} \text{...............} (6)$$

These equations are suggestive of useful approximations for mental checking as the work proceeds, both in design and construction.

Mr. Pretty. The writer takes this opportunity of thanking the author for bringing to his notice a paper of such unusual interest and wherein the difficulties met have been stated so frankly. It is, indeed, doubtful if any preliminary plans, without the experiences recorded, would have warranted such expensive methods of construction as those indicated in the final plans. To the professional onlooker, it represents a battle with natural forces in the cause of human progress, and one can only wish every success in the final results.

Mr. Baterden. J. R. BATERDEN,* ESQ. (by letter).—This is one of the most interesting papers written on such a subject. In Great Britain we are apt to be afraid to put forth records of a work which has not been wholly successful, but, as the writer has pointed out more than once, the record of one failure teaches us more than many successes, as it enables us to provide against a similar failure in the future.

It is seldom that an engineer has to undertake the building of a dry dock in such a depth of water and in such bad ground as in this case, and the paper opens up so many questions and gives so many details that it is difficult to make useful remarks on it.

The writer has not much faith in concrete deposited under water as a successful method of forming the foundation of the floor of a dry dock, especially with such heavy hydrostatic pressure as in the present instance. A large proportion of the concrete deposited by the tremie method (as the writer would expect) was not water-tight, and some of it was "generally of poor quality". The author toward the end of his paper agrees as to the want of certainty of good water-tight concrete being deposited under water. One of the risks of depositing concrete by the tremie or similar methods is that the heavier particles of the mixture tend to settle in the bottom, and, unless divers are employed, there is no opportunity of having the mortar thoroughly mixed with the stone, and thus consolidated, as in ordinary open work. The consequence is that in many cases a heap of stones accumulates with little or no mortar surrounding them. Where, as in this case, it was found necessary to deposit concrete in this manner, a richer mortar and of larger volume than is customary in ordinary work should be used.

The writer's experience in the use of "plums" in concrete for dry dock work has not been satisfactory. Theoretically, they are all right, but his difficulty has always been to ensure that the "plums" are satisfactorily placed. As long as they are completely surrounded by a sufficient thickness of water-tight mortar, the work is quite satisfactory, but the difficulty is to get this regulation adhered to. Even with the strictest supervision there is a risk of a heap of stones being deposited—for, of course, it is to the contractor's advantage to get

* Newcastle-upon-Tyne, England.

as many "plums" in the work as possible—or of their settling close together and not having sufficient mortar around them, thus forming a sieve for water to penetrate. The extra weight given to the concrete by their use is very little; the saving of cost is certainly appreciable. Where "plums" are used, the stone ingredients of the mixture should not be more than would go through a sieve of, say, from 1 to $1\frac{1}{2}$ in. square. Mr Baterden.

The writer quite agrees with the author's statement that mortar mixtures for water-tight concrete should be not more than 1 part of cement to 2 parts of sand. This produces a volume of about 2.1, so that the volume of ingredients for the concrete, allowing 45% for voids, should not be more than $3\frac{1}{2}$, which will allow a safe margin of mortar to fill the interstices and surround the stones; if he had a Government behind him, the writer would feel inclined to make the proportion of broken stone only 3, thus ensuring a harder and more water-tight concrete.

It would appear from the revised plans, that the idea of an intermediate caisson has been abandoned; this arrangement, in the writer's experience, has not been worth the extra cost.

It is a question whether a dry dock, 1 000 ft. long, is required in such a location. It is very unlikely that such a length will ever be required for Admiralty purposes; and, although it is true that the Panama locks are more than 1 000 ft. long, their case is different, as they provide for allowing two or possibly four of the present-day large vessels to pass through at the same time, as well as for the extreme length and beam of ships of the future. The 1 000-ft. ships of the future will be comparatively few, and every 50 ft. of shortening of dock, in ground such as that at Pearl Harbor, means a considerable saving.

The particulars given of bearing power and adhesion of piles in the ground are very interesting. As an instance of the weights which piles will carry, even in very soft ground, the writer, many years ago, supervised the test on four piles, 3 ft. from center to center, driven 25 ft. into the ground, and having on top a platform 6 ft. square to carry the load. The foundation at this place consisted of soft mud and muddy sand for a depth of about 180 ft. A load of 72 tons was placed on the platform, 18 tons per pile, which was much more than the piles in the adjoining ground were to carry. The following was the total settlement of each of the four piles, 6 days after the load had been placed on the platform: 1.08 in., 1.32 in., 3.96 in., and 3.12 in.

The greatest loading test on a single pile, of which the writer is aware, was one supervised by himself more than 20 years ago to test the ground for a battleship berth. The pile was of sawn pitch pine, 42 ft. long, about 12 in. square, and driven 37 ft. into the ground, which consisted of clays of varying degrees of stiffness, finishing in

Mr. Baterden.

a soft leafy clay. The weight of the hammer used was 5 600 lb., its fall was 5 ft., and the finishing sets were $\frac{7}{8}$ in. This pile actually carried 97 tons before commencing to sink, and then sank quickly and steadily.

It is a moot point whether piling is necessary as a foundation for a concrete dry dock, even in soft ground. The writer has had experience in dry dock construction in the worst of ground, where no piles were used, and the result was quite satisfactory. His experience with the jet process of pile sinking in mixed ground has been much the same as that of the author, and he believes it is only satisfactory where the subsoil is wholly in sand or fine material of a uniform character.

The new designs and method of carrying out the work might be called heroic, and appear to have been carefully thought out. The straight girder, as decided on, is certainly the best. Whatever slight advantages might be gained in strength by the arched form, and the lesser quantity of concrete required, would be more than counterbalanced by the almost impossibility of getting the required curved profile dredged accurately in such a depth of water. The writer hopes, and quite believes, that the work will be brought to a satisfactory conclusion.

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PAPERS AND DISCUSSIONS

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THE HYDRAULIC JUMP, IN OPEN-CHANNEL FLOW AT HIGH VELOCITY

Discussion.*

By Messrs. B. F. Groat, H. B. Muckleston, and Frederic P. Stearns.

B. F. Groat,† M. Am. Soc. C. E. (by letter).—The author has pointed out a principle which has commanded little or no attention. He has made clear the relations that control the limits of level within which the water in any channel must flow. Consequently, these relations control the upper limit of the possible rise of the hydraulic jump, although the effects of the channel and spillway conditions and, in particular, as Mr. Kennison indicates, the losses due to expanding cross-section, are still to be determined. Mr. Groat.

It appears that in any channel of usual shape there are two levels at which the water may flow with the same content of energy. Thus, so far as energy content is concerned, the water surface may change from one to the other of these levels without energy from any source other than that of the stream itself.

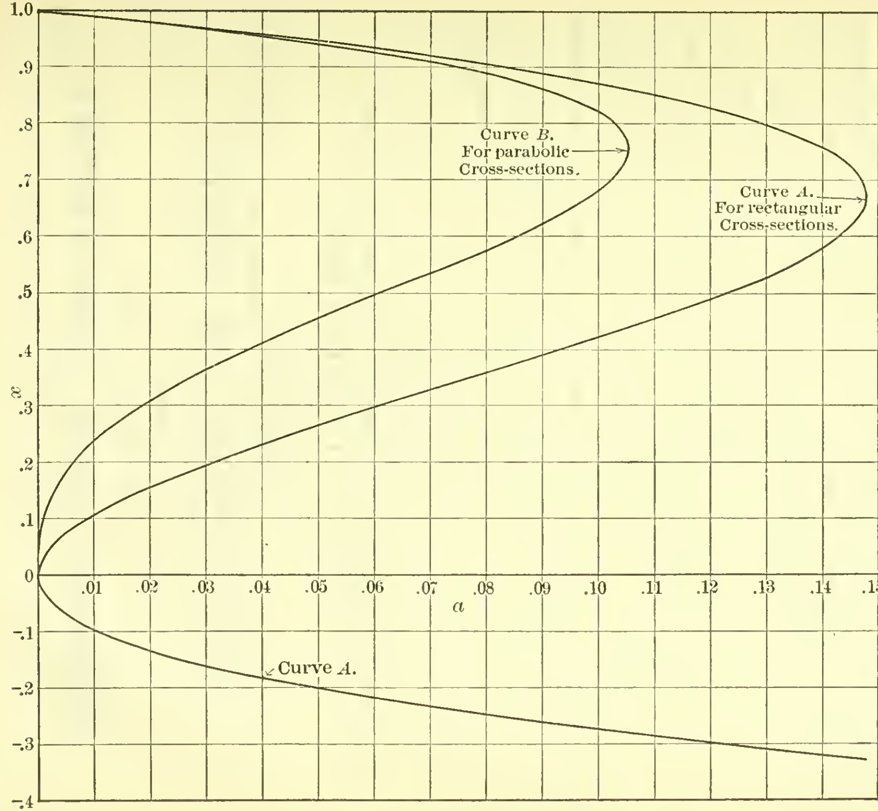
It also appears in the discussion, and this is clearly shown by Fig. 14, that the energy of the stream is always less for levels intermediate to the complementary levels, previously mentioned, than it is at either of them, and that it is greater outside of, than it is within, these limits. Therefore, the water cannot flow above the higher, or below the lower, of the complementary levels.

The great question is: What must happen at points between the two levels, at any of which there is a surplus of energy in the water? This surplus must appear in some form or another, the most likely being the energy which is absorbed by eddies. In other words, when

* This discussion (of the paper by Karl R. Kennison, Assoc. M. Am. Soc. C. E., published in September, 1915, *Proceedings*, and presented at the meeting of November 3d, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Pittsburgh, Pa.

Mr Groat.



Curve A.-Solution of the cubic equation $x^3-x^2+a=0$ for all values of a which furnish three real roots. This value of a is determined by the equation $a=\frac{Q^2}{2\,g\,H^3}$ where Q is the discharge over one foot width of a dam and H the total hydrostatic head on the dam. A positive root of the equation represents the ratio of the theoretical depth below the dam to the total hydrostatic head, that is $x=d\div H$. The following limitations are noteworthy:

$$0 < a < \tfrac{4}{27} = \tfrac{2^2}{3^3}$$
$$-\tfrac{1}{3} < x_1 < 0$$
$$0 < x_2 < \tfrac{2}{3}$$
$$\tfrac{2}{3} < x_3 < 1$$

x_1, x_2, x_3 being the three roots of the equation, only one of which is negative for any admissible value of a.

Curve B.-Solution of the biquadratic equation $x^4-x^3+a=0$ for all values of a which furnish two real positive roots. These values of a are determined by the equation, $a=\frac{Q^2}{2\,g\,k^2H^4}$, where $A=Kd^{3/2}$, A=area of cross-section of parabolic contour, d= the maximum depth above the bottom, k=a constant, and H, as above, is the total hydrostatic head on the dam. A positive root of the equation is the ratio of the depth below the dam to the total hydrostatic head on the dam, or $x=d\div H$ as above.

The limitation are:

$$0 < a < \tfrac{27}{256} = \tfrac{3^3}{4^4}$$
$$0 < x_2 < \tfrac{3}{4}$$
$$\tfrac{3}{4} < x_3 < 1$$

x_2 and x_3 being the two positive roots involved. Q is the total discharge in the case of Curve B.

FIG. 14.

the water surface is at either complementary level, does it not tend to move toward that intermediate level where the energy is a minimum? In the case of rectangular cross-sections, this level is at two-thirds the total energy head of the water; parabolic sections require three-fourths of the total energy head. At the foot of a dam the water may tend to rise to this level, but the rapid absorption of energy prevents it, as well as the fact that the shape of the channel below would not support such a flow in equilibrium, or there is no control section to support such a flow. Mr. Groat.

When the water surface is intermediate to the two complementary levels, it is certain that the formulas herein used do not apply, that is, the flow is not of the character supposed. In particular, the velocity is not uniform in the section, and there may be negative velocities, all of which destroy the value of the formulas for anything like accurate indications.

The height of the standing wave, for example, must depend on the shape, size, and character of the channel below the dam and also on the character of flow over the dam, as, for example, when changes of condition of gate-opening occur in the case of arch dams provided with sluice-gates at the crest.

As the writer had the privilege of examining Mr. Kennison's treatment in connection with an important investigation, it may not be amiss to append some further study of the subject.

Fig. 14 solves problems relative to the two complementary levels for all channels having rectangular or parabolic cross-sections.

Using the author's notation, let:

$$a = \frac{Q^2}{2\,g\,H^3}, \text{ for rectangular sections;}$$

$$\text{and } a = \frac{Q^2}{2\,g\,K^2\,H^4}, \text{ for parabolic sections;}$$

where, in the latter case,

$$\text{the section area, } A = K\,x^{\frac{3}{2}}.$$

Let it be required, for example, to solve, by Fig. 14, the problem proposed by Mr. Kennison on page 1709* of his paper.

We have: $H = 10$ ft., $Q = 50$ cu. ft. per sec., and, therefore, $a = 50^2 \div 2\,g \times 10^3 = 0.0388$ for a rectangular section.

With this value of a, the corresponding values of the three ordinates to Curve A on Fig. 14, are 0.958, 0.224, and — 0.181.

As the curve relates to unit head, the three ordinates must be multiplied by H (10 ft. in this case), to secure the values of the corresponding depths. Therefore, the three possible depths for a total energy head of 10 ft. are 9.58 ft., 2.24 ft., and — 1.81 ft., of which the

* *Proceedings*, Am. Soc. C. E., for September, 1915.

Mr. Groat. two positive values are the only ones sought. It will be seen that the results agree with the values deduced by Mr. Kennison by the solution of a cubic equation.

Curve B is used in exactly the same manner when the cross-section of the stream may be represented as the area of a parabola with its axis vertical and the vertex at the lowest point of the bed of the stream.

A better understanding of the subject and of the use of the curves will be gained by the following study of energy relations.

Rectangular Sections.—Let x be the depth in the channel, supposed to be rectangular, so that we need consider only 1 ft. of the width. The velocity is also supposed to be uniform, in order to simplify the discussion. Let v be the velocity and h the energy head. Then, the energy head at any point is given by

$$h = x + \frac{v^2}{2g} \quad \text{.........................(1)}$$

If q is the discharge, we then have:

$$q = v\,x \quad \text{.........................(2)}$$

or,

$$v = \frac{q}{x}$$

whence,

$$x + \frac{q^2}{2\,g\,x^2} = h \quad \text{.....................(3)}$$

or,

$$x^3 - h\,x^2 + \frac{q^2}{2\,g} = 0.$$

If, in Equation (3), we put

$$a = q^2 \div 2\,g\,h^3 \quad \text{.....................(4)}$$

and

$$r = x \div h$$

there results,

$$r^3 - r^2 + a = 0 \quad \text{.....................(5)}$$

in which a is a variable depending on the value of x, that is, on r. In other words, the energy head depends on the depth of the water in the section and varies in a sense contrary to that of the variation of a. Thus, when a is a maximum, the energy head is a minimum, and *vice versa,* if we suppose the discharge constant. If h is supposed to be constant, then the discharge is a maximum simultaneously with a.

If the discharge is supposed to be constant, then it is clear that there are two levels, as Mr. Kennison has pointed out, either of which can be maintained with the same energy head, h, as it can be shown that two of the roots of the cubic equations, (3), or (5), are positive, and the third is negative.

Let it be required to determine the relation between the two positive roots of Equation (5) so that, if one of the two possible levels of the water is known, the other can be found.

In order to do this, let n be the numerical value of the negative root of Equation (5). Then, by the theory of equations: Mr. Groat.

$$\frac{r^3 - r^2 + a}{r + n} \text{ is an exact quotient} \qquad (6)$$

that is,

$$r^2 - (n + 1)\, r + n\, (n + 1) = 0 \qquad (7)$$

from which,

$$r = \frac{1 + n \pm \sqrt{(1 + n)\ \ (1 - 3\,n)}}{2} \qquad (8)$$

This last equation determines the two positive roots in terms of the numerical value of the negative root. Table 1 has been computed by it, for the complete range of applicable values of r.

TABLE 1.—Values of r, for the Equation,

$$r^3 - r^2 + a = 0.$$

$n^3 + n^2 = a$	n	$\sqrt{(1+n)\,(1-3\,n)}$	r_1	r_2
0.000408	0.03	0.9792	0.9996	0.0204
0.001664	0.04	0 9567	0.9984	0.0417
0.003816	0.06	0.9323	0.9962	0.0639
0.006912	0.08	0.9060	0.9930	0.0870
0.011000	0.10	0.8775	0.9888	0.1113
0.016128	0.12	0.8466	0.9833	0.1367
0.02234	0.14	0.8131	0.9766	0.1635
0.02970	0.16	0.7767	0.9684	0.1917
0.03823	0.18	0.7367	0.9584	0.2217
0.04800	0.20	0.6928	0.9464	0.2536
0.05905	0.22	0.6441	0.9321	0.2880
0.07142	0.24	0.5892	0.9146	0.3254
0.08518	0.26	0.5265	0.8933	0.3668
0.10035	0.28	0.4525	0.8663	0.4138
0.11700	0.30	0.3606	0.8303	0.4697
0.13517	0.32	0.2298	0.7749	0.5451
4/27	1/3	0	2/3	2/3

It will be seen from Equation (8) that n is limited to positive values which do not exceed $\frac{1}{3}$. At the limit 0, $r = 0$, or, $r = 1$, while the value, $\frac{1}{3}$, gives two equal values of r, that is, $\frac{2}{3}$ is a double root. This double root corresponds to a maximum value of a, and, consequently, to a minimum value of the energy head or to a maximum value of the discharge for a given energy head.

By Table 1, the graph of Equation (8) has been plotted as Curve A, on Fig. 14, which exhibits very clearly all the properties which have been discussed and, in addition, that there is a double root, 0, which corresponds to a minimum value of a. This is also clearly shown by Equation (8).

An illustration of the use of the curve has already been given.

It is now desirable to express one of the positive roots of Equation (5) in terms of the other, so that, if the depth of water in a channel

Mr. Groat. is known, the complementary depth for the equal energy head can be determined by a simple formula.

In order to deduce this formula, observe that, if d_1 and d_2 are the two complementary depths, one of which, say, d_2, is known, then the two corresponding complementary roots of Equation (7) are $r_1 = d_1 \div h$, and $r_2 = d_2 \div h$, and they furnish, therefore, the following two equations:

$$\left. \begin{array}{l} r_1^2 - (n + 1)\, r_1 + n\,(n + 1) = 0 \\ r_2^2 - (n + 1)\, r_2 + n\,(n + 1) = 0 \end{array} \right\} \ldots\ldots\ldots\ldots\ldots (9)$$

As these equations must consist with each other, the relation between r_1 and r_2 may be deduced by eliminating n from them. This elimination may be effected most simply by taking the difference between the equations, thus:

$$r_1^2 - r_2^2 + (r_2 - r_1)\,(n + 1) = 0 \ldots\ldots\ldots\ldots (10)$$

from which,

$$n + 1 = r_1 + r_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

This relation might easily have been inferred from Equation (5), as the coefficient of r^2 is -1, that is,

$$n - r_1 - r_2 = -1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

is the negative sum of the roots.

Substituting the values of n and $n + 1$ in either form of Equation (9), it is easy to show that

$$r_1^2 + (r_2 - 1)\, r_1 + (r_2^2 - r_2) = 0 \ldots\ldots\ldots\ldots (13)$$

which gives r_1 in terms of r_2 thus:

$$r_1 = \frac{(1 - r_2) + \sqrt{(1 - r_2)\,(1 + 3\,r_2)}}{2} \ldots\ldots\ldots\ldots (14)$$

the negative radical being omitted from the numerator as both d_1 and d_2, and, therefore, both the roots must be positive.

It will be more convenient, however, to have d_1 and d_2 expressed directly as functions of each other. This may be accomplished by substituting for r_1 and r_2 in Equation (14), thus

$$\frac{d_1}{h} = \frac{1 - \frac{d_2}{h} + \sqrt{\left(1 - \frac{d_2}{h}\right)\left(1 + \frac{3\,d_2}{h}\right)}}{2} \ldots\ldots\ldots (15)$$

from which,

$$d_1 = \frac{h - d_2 + \sqrt{(h - d_2)\,(h + 3\,d_2)}}{2} \ldots\ldots\ldots\ldots (16)$$

and, by observing that,

$$h = d_2 + \frac{v_2^2}{2\,g},$$

we may easily obtain, Mr. Groat.

$$d_1 = \frac{\frac{v_2^2}{2\,g} + \sqrt{\frac{v_2^2}{2\,g}\left(\frac{v_2^2}{2\,g} + 4\,d_2\right)}}{2} \qquad (17)$$

or,

$$d_1 = \frac{v_2^2}{2\,g}\left(\frac{1}{2} + \sqrt{\frac{1}{4} + R}\right)$$

where,

$$R = \frac{d_2}{\frac{v_2^2}{2\,g}} \qquad (18)$$

R thus being the ratio of potential to velocity head.

Thus, d_2 being 2.24, and $v_2^2 \div 2\,g$ being 7.76, we should have $R = 2.24 \div 7.76 = 0.289$, from which,

$$d_1 = 7.76\,(0.5 + \sqrt{0.539}) = 9.58 \qquad (19)$$

which agrees with Mr. Kennison's value on page 1709.* If we take $d_2 = 9.58$ and $v_2^2 \div 2\,g = 0.42$, in Equation (17), it will be found that $d_1 = 2.24$ and $v_1^2 \div 2\,g = 7.76$, a necessary result due to the reciprocal relation between d_1 and d_2. (See Equations (20).)

Equation (17) may be deduced directly by solving

$$d_1 + \frac{v_1^2}{2\,g} = d_2 + \frac{v_2^2}{2\,g} \qquad (20)$$

and,

$$d_1\,v_1 = d_2\,v_2$$

for d_1 in terms of d_2 after cancelling the factor, $d_1 - d_2$.

Now, it is easy to see that $0 < r_1 < \frac{2}{3}$ and $\frac{2}{3} < r_2 < 1$, and that either the equation, $r^3 - r^2 + a = 0$, or Fig. 14, showing the trace of the relation between a and r, shows that a is greater for all values of r between r_1 and r_2 than it is for r_1 or r_2; that is, h, the energy head of the flowing water, for all depths between d_1 and d_2, is less than it is at either of these limits, and the energy is greater at depths below d_1 or above d_2. Hence, the flow of the water is limited to some depth, x, where $d_1 < x < d_2$. That is, the water may flow at any level between d_1 and d_2, but not outside these limits. This, of course, supposes a rectangular channel and that a sufficient amount of energy is absorbed where the level is at neither limit.

Parabolic Cross-Sections.—The foregoing method of analysis may be extended to shapes of section other than rectangular with corresponding modifications. Suppose, for example, that the section area is connected to the depth by

$$A = k\,x^{\frac{3}{2}} \qquad (21)$$

* *Proceedings,* Am. Soc. C. E., for September, 1915.

Mr. Groat. A being the area, x the depth, as before, and k a constant.

Then,
$$h = x + \frac{v^2}{2\,g} \quad \text{.......(22)}$$

$$q = A\,v \quad \text{.......(23)}$$

and,
$$v = \frac{q}{A} = \frac{q}{k\,x^{\frac{3}{2}}} \quad \text{.......(24)}$$

Therefore,
$$h = x + \frac{q^2}{2\,g\,k^2\,x^3} \quad \text{.......(25)}$$

$$x^4 - h\,x^3 + \frac{q^2}{2\,g\,k^2} = 0 \quad \text{.......(26)}$$

from which,
$$r^4 - r^3 + \frac{q^2}{2\,g\,k^2\,h^4} = 0$$

or,
$$r^4 - r^3 + a = 0 \quad \text{.......(27)}$$

where,
$$a = \frac{q^2}{2\,g\,k^2\,h^4} \quad \text{.......(28)}$$

and,
$$r = x \div h.$$

It is then easy to show, by reasoning exactly as in the case of a rectangular channel, that a is a maximum and H a minimum when $r = \frac{3}{4}$, that is, when the depth is three-fourths of the total energy head. The graph of the equation for the applicable values of the roots may be constructed from Table 2.

TABLE 2.—VALUES OF r FOR THE EQUATION,

$$r^4 - r^3 + a = 0.$$

r	r^3	r^4	a	Remarks.
0	0	0	0	
0.1	0.0010	0.0001	0.0009	
0.2	0.0080	0.0016	0.0064	
0.3	0.0270	0.0081	0.0189	
0.4	0.0640	0.0256	0.0384	
0.5	0.1250	0.0625	0.0625	
0.6	0.2160	0.1296	0.0864	
0.7	0.3430	0.2401	0.1029	
¾	0.4219	0.3164	0.1055*	$(\frac{3}{4})^3 \cdot (1 - \frac{3}{4})$* = maximum value of $a = \frac{3^3}{4^4}$.
0.8	0.5120	0.4096	0.1024	
0.9	0.7290	0.6561	0.0729	
1.0	1	1	0	

The curve is marked B on Fig. 14. Only two positive roots are applicable, there being a pair of imaginary roots for the admissible range in value of a, the maximum of which is $3^3 \div 4^4 = 0.1055$, when $r = \frac{3}{4}$. Mr. Groat.

Let it be required, as in the former case, to deduce the relation between the two positive roots, and thus a formula for determining the second possible water level when the first is known, supposing that no losses of energy occur in the change.

This may be accomplished by a procedure analogous to that adopted in the case of rectangular cross-sections, but the following will probably be found more expeditious:

Let d_1 and d_2 be the two possible levels. We then have,

$$d_1 + \frac{v_1^2}{2\,g} = d_2 + \frac{v_2^2}{2\,g} \quad \text{(29)}$$

but,

$$v^2 = q^2 \div A^2 = q^2 \div K^2\,x^3 \quad \text{(30)}$$

therefore,

$$d_1 - d_2 = \frac{q^2}{2\,g\,K^2}\left(\frac{1}{d_2^3} - \frac{1}{d_1^3}\right)$$

$$= \frac{q^2}{2\,g\,K^2}\;\frac{d_1^3 - d_2^3}{d_1^3\,d_2^3} \quad \text{(31)}$$

from which we easily find

$$1 = \frac{q^2}{2\,g\,K^2\,h^4}\;\frac{d_1}{r_1^4}\;\frac{d_1^2 + d_2\,d_1 + d_2^2}{d_2^3} \quad \text{(32)}$$

Thus, by Equation (28),

$$1 = \frac{a}{r_1^4}\,(R^3 + R^2 + R) \quad \text{(33)}$$

or,

$$R^3 + R^2 + R - b = 0,$$

where R is the ratio of the two applicable roots, and,

$$b = \frac{r_1^4}{a} = \frac{d_1}{\frac{v_1^2}{2\,g}} = \frac{r_1}{1 - r_1} \quad \text{(34)}$$

It may be shown that two of the roots are imaginary for all applicable values of b, that is, of a and r_1, so that there will be only one solution, which is given by,

$$A = \frac{7}{54} + \frac{b}{2}$$

$$B = \sqrt{A^2 + \left(\frac{2}{9}\right)^3}$$

$$R = (A + B)^{\frac{1}{3}} + (A - B)^{\frac{1}{3}} - \frac{1}{3} \quad \text{(35)}$$

Mr. Groat.

For example, if $r_1 = 0.45$, then $b = 45 \div 55 = 9 \div 11 = 0.8181$ and $a = 0.050119$, from which,

$$A = \frac{7}{54} + \frac{b}{2} = 0.538720$$

$$B = \sqrt{A^2 + \left(\frac{2}{9}\right)} = 0.548811$$

$$A + B = 1.087531$$

$$A - B = -0.010091$$

and

$$R = 1.087531^{\frac{1}{3}} - 0.010091^{\frac{1}{3}} - \frac{1}{3}$$

$$= 1.028365 - 0.216094 - \frac{1}{3}$$

$$= 0.47894.$$

Therefore, we must have,

$$r_2 = \frac{r_1}{R} = \frac{0.45}{0.47894} = 0.93957,$$

the correctness of which may be verified by reference to Fig. 14, for $a = 0.0501$. It will be seen that the two values of r, for this value of a, agree with the values just computed.

It is now easy to see how any shape of cross-section can be treated in a manner similar to the foregoing, whether the section is, or is not, expressible in algebraic form. It is also clear from the foregoing treatment, that the complementary levels vary relatively for different shapes of section, and that the elevation of the water surface in control sections also depends on the shape as well as on the magnitude of the sections, being two-thirds of the total energy head for rectangular sections and three-quarters of the energy head for parabolic sections. The jump, likewise, is governed by the shape of the channel.

An interesting question arises in the case of cross-sections other than rectangular. If individual widths are treated separately, it will be found that the corresponding complementary levels vary according to location across the stream. That is, theoretically, the water surface should present a curved transverse profile. May this not be one of the prime causes of much of the disturbance which occurs in streams of irregular cross-section?

The writer is of the opinion that we wander too far from the domain of theoretical hydrodynamics when we apply equilibrium equations to finite masses and volumes of water. This can be done with exactness only when we are considering rigid bodies. The difficulty has always been that of integrating the differential equations throughout the volumes and over the bounding surfaces of the masses of water involved. However, much progress has been made and still more remains

for the future. In time, we shall understand the nature of the losses which occur by reason of expanding sections in draft-tubes and in the hydraulic jump. Mathematics and experimental physics must join hands in the work. Fortunately, the density is practically constant, which simplifies matters considerably, but our knowledge of the effects of viscosity is very limited. Mr. Groat.

Varied examples of the hydraulic jump may be seen at most sea beaches where the slope is not too steep. A favorable condition occurs when the slope is moderate for a considerable distance up the beach followed by a steeper slope above. The steeper slope aids in accelerating the water receding down the beach. When the receding water attains the right velocity with respect to an incoming wave, the jump occurs. These jumps seldom exceed 1 ft. in height, but present an interesting variety of phenomena.

H. B. MUCKLESTON,* M. AM. SOC. C. E. (by letter).—This interesting paper should be very valuable to those who have to deal with the phenomena discussed. Two instances are cited where the question proved important for widely different reasons, and many others will readily occur to the reader. Mr. Muckleston.

It will be understood that the formulas presented are like all other hydraulic formulas founded on incomplete theory, or probably—put in a better way—on theory founded on incomplete or erroneous assumptions, and that in practice they will have to be modified by a coefficient.

The author points out one case, in the application to weir discharge, where the theoretical formula must be modified to agree with the results of experiment.

On page 1709,† the author makes the convenient but inaccurate assumption that the velocity head of a flowing stream is equal to $\frac{V^2}{2g}$, where V is equal to the discharge divided by the cross-sectional area. This is never quite true, and in extreme cases may be very far from the truth. Assume, for example, a stream divided into two parts, and that the mean velocity is 14 ft. per sec. in the upper part and 2 ft. per sec. in the lower. On the author's assumption, the velocity head for the whole would be 1.0 ft., nearly. Taking the two values separately, and using the mean of the values thus found, it would be about 1.56 ft.

The case is extreme, and probably impossible, but it serves to show that the assumption is not correct.

Referring now to the case of the Bassano Dam, cited by the author and illustrated by Fig. 2. This dam is about 38 ft. high above the down-stream apron, and may discharge about 13 ft. deep over the

* Calgary, Alberta, Canada.

† *Proceedings*, Am. Soc. C. E., for September, 1915.

Mr. Muckleston. crest. The concrete apron below the ogee is about 50 ft. long, and it was feared that at certain stages the jump or standing wave might be pushed down stream to such a distance that it would occur on the unprotected river bed, which in this case is gravel overlying clay. In order to insure that this should not occur, on the recommendation of Mr. John R. Freeman, two rows of baffle piers were built on the ogee, as described by the author. The dam has now been through two seasons and one very large flood, and it may be interesting to give the results of close observation of the behavior of these baffles.

Owing to the presence of two large under-sluices near the center of the dam at apron level, a length of 60 ft. was not equipped with them, and another length next to the east abutment was left unprovided on account of a turbine outlet. There was thus an opportunity to observe closely, especially at moderate stages, the results of building these baffles. It should be noticed that the level of the stream below the dam rises very much faster than does the pond above, and that, in consequence, the depth of cushion increases very much faster than the discharge. For instance, with about 4 ft. on the crest, the depth on the apron is about the same, for 10 ft. on the crest, the depth on the apron was about 18 ft.

It was noticed that at very low stages, say, less than 3 ft., the stream striking the baffles was so thin that practically all the energy was absorbed by impact. As the stage increased, the effect of the baffles was seen in a series of jets which did not impinge on one another, but, on the contrary, were thrown out on the apron, especially the jets from the upper row. This effect increased in volume up to a stage of about 6.5 ft., when the depth on the carpet began to mask the baffles, and the violence of the jets began to decrease, until at a stage of about 8.5 ft. the action of the baffles was completely obscured by the water-cushion.

Considering that part where no baffles were provided, it was observed that for stages up to about 3 ft. the jump occurred on the ogee. (The stage varies because the depth of cushion for any given stage varies somewhat on account of the condition of the stream bed below.) From this up to about 7 ft. the jump is pushed down stream for a short distance, estimated at about 20 ft., after which it begins to return. Figs. 15 and 16, reproduced from photographs taken from nearly the same point, illustrate the action at two different stages. Fig. 15 shows the dam at about 5.5 ft. on the crest. The unequipped length is very evident, as is also the action of the baffles in throwing out the almost horizontal jets. Fig. 16, showing the dam with 9.5 ft. on the crest, is specially interesting, when compared with Fig. 15. The section of the dam not equipped is not at all evident, and cannot be located except by counting the bridge piers. Looking down from the bridge, there did not appear to be any difference in the action of the two parts.

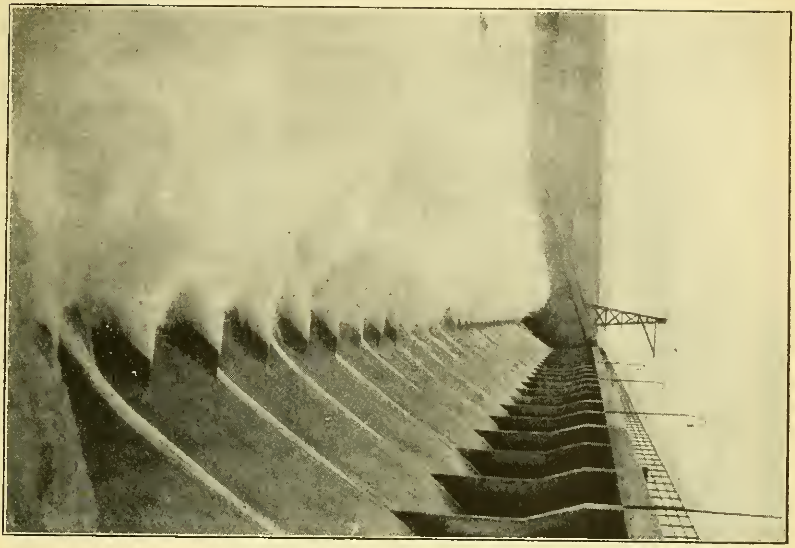


FIG. 15.—BASSANO DAM: OVERFLOW WITH 5.5 FT. ON THE CREST.

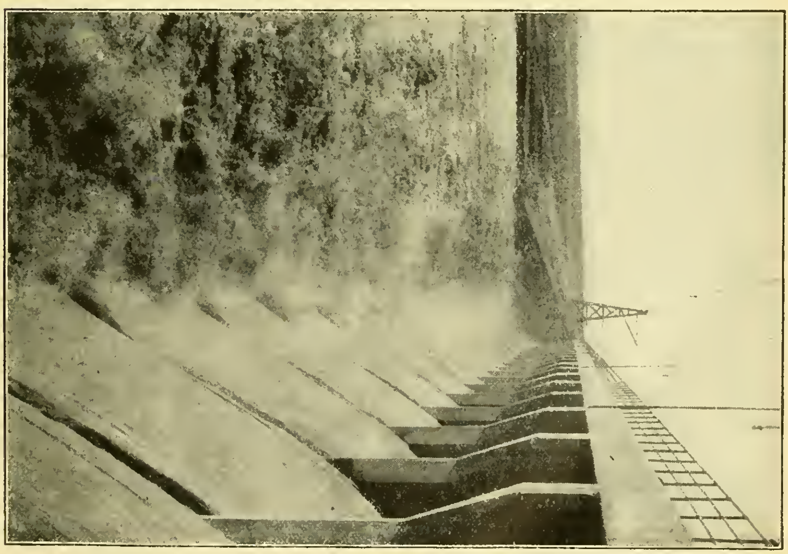


FIG. 16.—BASSANO DAM: OVERFLOW WITH 9.5 FT. ON THE CREST.

The author's conclusions are interesting and valuable, but there is a further conclusion which he did not draw. The writer is of the opinion that the ogee is the wrong method of treating a spillway dam, and that any device which may be added to minimize its bad effects is only a palliative and not a cure. A straight overfall into a deep water-cushion is Nature's method, and, if adopted in the work of Man, no evil effects from jump, back roll, standing wave, or whatever it may be called, need be feared. As far as the writer knows, this has never been investigated, except as regards the effects on the floor of the cushion. Many years ago, Col. Dyas experimented in India by sending thin glass bottles over falls, and observed that they were not broken when the depth of the cushion exceeded $h^{\frac{1}{2}}\ d^{\frac{1}{3}}$, where h is the height of the fall and d is the depth on the crest. For design, this was modified to one-half or one-third of the depth thus found, depending on the material of the floor. Mr. Muckleston.

Frederic P. Stearns,* Past-President, Am. Soc. C. E. (by letter).— This paper is a very valuable contribution on a subject which has rarely been discussed, but is of high practical importance. The treatment of the subject by the author from a theoretical and mathematical standpoint is so complete that the writer can add nothing of value in these respects. Mr. Stearns.

The author makes this statement:

"A knowledge of the hydraulic principles involved [in the hydraulic jump] should enable destructive high velocities and turbulence to be avoided, or intelligently provided for, in the design of flumes, dam foundations, etc."

Elsewhere in the paper, also, he calls attention to the practical dangers of permitting water to flow with high velocity at the lower "alternative stage". It seems desirable that the dangers involved in such high velocities and the methods by which they may be prevented should be still further emphasized.

In the writer's experience, damage occurred to a wooden flume, referred to in the paper, because the water flowed at the lower "alternative stage" for a long distance before the hydraulic jump occurred. The flume, 40 ft. wide, 16 ft. high at its upper end, and 13 ft. at the lower end, was built to carry the water of a stream past a masonry dam during the construction of its lower portion. It was built of planed lumber, with no obstructions on the inside. Its total length was about 700 ft. At a point 380 ft. from its upper end there was an angle in the flume, the deflection from a straight line being 8.5 degrees. The approach was flared, to prevent losses of head due to contraction at the entrance.

* Boston, Mass.

Mr. Stearns.

The upper end of the flume passed through an earth coffer-dam, and, on the center line of the latter, 30 ft. below the entrance to the flume, provision was made for removable stop-logs, which were kept in place except during floods to maintain a water supply through an aqueduct. The plans provided for a coffer-dam across the valley at the lower end of the flume, but at the time of the floods referred to subsequently this dam had not been built.

In designing the flume, the maximum quantity of water to be discharged during the greatest floods was assumed to be 9 000 cu. ft. per sec., and it was expected that with this discharge there would be a loss of head at the entrance of about 5 ft., and that the flume would run nearly full.

The writer recognized at that time that immediately below where the drop in the surface occurred, at the entrance to the flume, the water surface might be lower than at points farther down stream, but there was nothing in his experience or that he had gleaned from the experience of others to lead him to think that the low-water surface would persist for any long distance.

A flood substantially equal to that assumed in designing the flume occurred, and the flume operated substantially in accordance with the expectations at the time it was designed, as there was a moderate loss of head at the entrance and the flume ran nearly full.

Soon afterward, a second flood occurred, somewhat smaller than the first, but the water, instead of nearly filling the flume, ran with high velocity with a depth of less than half the height of the flume, until it reached the slight angle 380 ft. from its upper end. The hydraulic jump then took place, not all at once, but rather gradually, in the next 275 ft. The water on the outside of the flume, corresponding as it did to the height of water at its lower end, was from 1 to 2 ft. higher than the water inside, where it was flowing at the low stage, thereby causing an upward pressure on the bottom sufficient to lift the flume off its foundation for a considerable part of its length, notwithstanding the fact that it was anchored in some places to the ledge and in other places was fastened to anchors buried 3 ft. below the surface of the ground which supported the sills.

Just why the water flowed at such different depths in the flume in the two floods is not definitely known, but it is probable that it was due to the removal of all the stop-logs and other obstructions at the time of the first flood and to leaving a part of these logs in place during the second flood. The author illustrates the conditions toward the upper end of this flume in Fig. 10.

The damage done to the flume might have been avoided had some comparatively small obstructions been placed in its bottom toward the upper end, so as to cause the hydraulic jump to take place there. Very few flumes are built where the damage which occurred in this case

would be likely to occur, and the incident has been fully described in order to illustrate a cause of danger which is frequently present below spillway dams. Mr. Stearns.

Taking now the case of a high spillway dam, like that illustrated by Fig. 12, the water attains a high velocity as the result of the fall from the reservoir, and, owing to the easy curve at the bottom of the dam, is projected horizontally along an apron with the velocity still high, and a correspondingly small depth, but at a greater or less distance below the dam the hydraulic jump occurs, and down stream from this jump the water is much higher than above it.

Assuming, in a case like that described and illustrated, that there is a level concrete apron without vents to relieve the pressure beneath it, over which the water moves with high velocity, and that the hydraulic jump takes place at the lower edge of the apron, it will then seem obvious that the water pressure beneath the apron will be that due to the comparatively high level of the water down stream from the hydraulic jump, although this may be somewhat increased by the pressure transmitted from the reservoir itself through the strata beneath the dam. In other words, the pressure tending to lift the concrete apron will not be less than that represented by the difference of level between the surface of the swiftly moving water up stream from the jump and the slower moving water down stream from it. If the excess of pressure beneath the apron is greater than the weight of the apron, it will lift.

The author's Fig. 12, based on a spillway dam 185 ft. high, is, so he states, "more or less arbitrarily drawn", but it is sufficiently near what may occur in practice to be used for the purpose of illustration. By scaling from the illustration, the jump from the low stage to the high one is 34 ft., and if the level portion below the dam was a concrete apron without vents, a thickness of fully 23 ft. would be required to balance this net head, assuming the concrete when immersed in water to weigh 90 lb. per cu. ft.

It is frequently the case that there is trouble below spillway dams as the result of heavy floods, and an important factor in causing such trouble is a failure to stop the high velocity close to the foot of the dam.

High velocity below a dam may cause trouble in two ways:

(*a*)—In the manner already described, owing to the small pressure of the swiftly moving water on the top of a nearly level apron, or on the surface of a ledge, in comparison with the pressure under the apron, or in the seams of the ledge when these communicate with the slowly moving water at a higher level below the jump;

(*b*)—By reason of the swiftly moving water impinging upon a crack or seam facing up stream, thereby causing in the crack or seam the increased pressure due to the velocity of the water, much in the

Mr. Stearns. same way that increased pressure or head is produced in a Pitot tube used in a swiftly moving stream when the orifice faces the current.

From one or the other or both of these causes, aprons may be lifted, great masses of rock lying in horizontal strata may be displaced, and holes may be dug in the rock below important spillway dams to an extent that seems nearly incredible to those who have not witnessed the results or studied the forces acting to produce such movements.

The author makes this statement, on page 1718:*

"The fact remains that the destructive energy due to the drop over a spillway dam is a definite, fixed quantity, regardless of the presence or absence of any hydraulic jump."

The writer agrees with this statement, if the word "destructive" is omitted.

There is a definite amount of energy due to the fall of the water, but it may or may not be destructive. Water may fall through a turbine, converting the energy due to the drop into other forms of energy, and yet be in no wise destructive. Similarly, the energy due to the drop over a spillway dam may be converted into heat close to the foot of the dam, without being destructive, if the material upon which the water drops can be depended on to endure the shock and wear caused by the swiftly moving water and by all other substances which are carried by it over the dam.

The common practice of curving the bottom of a concrete dam, so as to throw the sheet of water out in a nearly horizontal direction with little loss of velocity, is one which seems to have had its origin either in the idea that this changing of the direction of the water would prevent erosion of the material below the dam, or that the material used in constructing the dam could not be depended on to stand the shock of the falling water and other substances. The use of this bottom curve has signally failed to prevent the erosion of rock and other material below a dam, and materials are now available for constructing dams, which under most circumstances can be depended on to endure the shock and wear resulting from the drop of the water in great quantities over a very high dam.

The writer believes that the policy in respect to the lower portion of spillway dams should be modified and that the high velocity should be destroyed at the point where the water reaches the bottom of the dam, and not be merely changed in direction.

The high-grade concrete which it is feasible to make at the present day, if used in sufficient thicknesses and amply reinforced with steel, can be depended on in nearly all cases to endure the shock and wear caused by water and all other substances which fall over a dam, and, if necessary, points subject to excessive shock and wear can be faced

* *Proceedings*, Am. Soc. C. E., for September, 1915.

with cast iron. The shock can be greatly lessened, where it is feasible to provide an adequate water-cushion. Mr. Stearns.

The method to be used in different cases will naturally vary. On page 1718* the author has referred to one method:

"The Bassano Dam of the Canadian Pacific Railroad Irrigation Project is equipped, after the recommendation of Mr. John R. Freeman, with two staggered rows of 'baffle-piers', shaped like snow-plows, pointed up stream, and designed to split up the high-velocity sheet of water before it can strike the bed of the stream, and throw one jet against another so that the energy will be absorbed as much as possible by eddies within the body of the down-stream pool, and not by tearing the foundations. These baffle-piers, backed up by a water-cushion, give assurance that the jump will start at the toe of the dam."

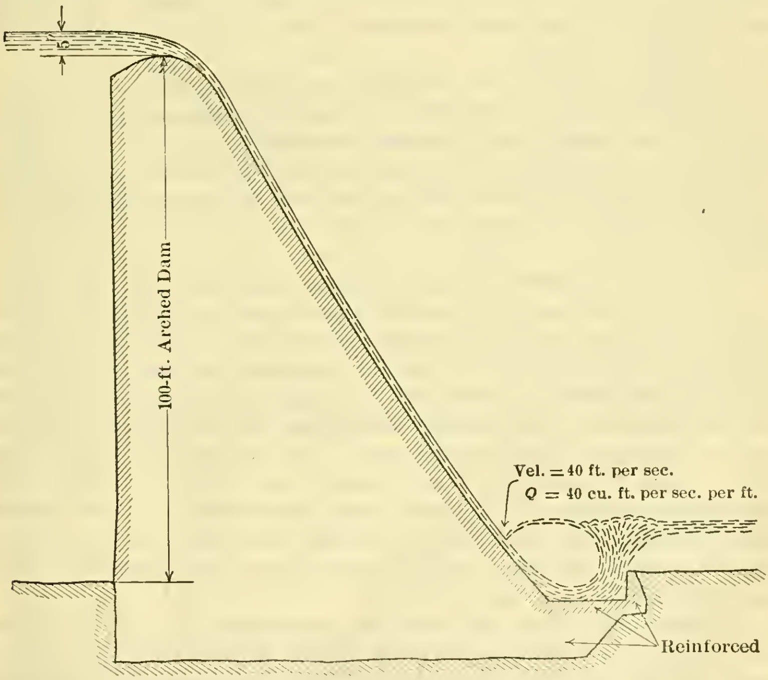


FIG. 17.

Fig. 2 is a view of this dam in action.

The writer, as already indicated, believes that, as a rule, a more complete destruction of the high velocity at the foot of the dam should be attempted. One method of doing this is indicated by Fig. 17, which represents (with slight modification) the arrangements provided at a spillway dam designed in part by the writer.

* *Proceedings*, Am. Soc. C. E., for September, 1915.

Mr. Stearns. It was necessary in this case, and it is frequently necessary in order to obtain a suitable foundation for a dam, to excavate to a considerable depth below the river bed, and if the down-stream face of the dam is continued to below the bed without a curve, and the excavation on the down-stream side is filled with concrete to a level below the bed, a large thickness of concrete upon which a sheet of water may strike is easily provided; also a water-cushion.

The sketch, Fig. 17, illustrates this method of construction. The concrete on the down-stream side is built to a level about 5 ft. below the river bed, and 15 ft. from the down-stream face a wall is built to the level of the river bed. By such an arrangement, there is a pool having a depth of 5 ft. in addition to the depth to which the water rises above the river bed with different river discharges. If, in the case of a big flood, the water in the river rises 10 ft., the sheet will fall into a pool 15 ft. deep.

In order to visualize the conditions existing at such a dam during a maximum flood, let it be assumed that the maximum depth on the crest will be substantially 5 ft. and the quantity of water per linear foot 40 cu. ft. per sec. The velocity of the descending water, where it enters the pool at the foot of the dam, will be between 30 and 60 ft. per sec., making the depth of the sheet at this place between 16 and 8 in.

To use an intermediate figure between these limits, it may be assumed for the purpose of discussion that the sheet will be 1 ft. thick and have a velocity of 40 ft. per sec. Water with this velocity entering the pool at the face of the dam would obviously continue to flow down the face to the bottom of the pool. It would entrain some of the water through which it passed, thereby decreasing the velocity and increasing the thickness of the moving mass. On reaching the bottom, the water would be deflected down stream, and would flow with still diminishing velocity, for the most part horizontally, until its motion was retarded by the wall 15 ft. from the dam; thence it would flow upward, most of the water turning down stream, but a part of it turning up stream to take the place of the water entrained.

It seems quite clear that with the proportions thus given for the depth of the pool and the thickness of the flowing sheet, the energy due to the drop of 100 ft. would be thoroughly taken care of in the pool within 15 or 20 ft. of the bottom of the down-stream slope, without destructive action, and that at points farther down stream there would be no high velocity due to the fall of the water over the dam.

If there was any doubt as to the action thus described taking place in a pool at the base of a dam, the further precaution might be taken of building occasional projections, from the face of the dam, irregularly spaced at different distances below the flood level in the river. Such projections should be strong enough to resist all shocks, and

would deflect parts of the sheet of water, making it impossible for it to flow at the lower "alternative stage". The confused currents caused by such projections would quickly entrain large quantities of the water in a pool, thereby producing a fairly regular flow within a very short distance from the foot of the dam. Mr. Stearns.

The design of the spillway at the Gatun Dam at the Panama Canal contains features to be commended, especially in cases where the quantity of water per linear foot of spillway is very large. In this case, the raising of large gates on the spillway crest, 18 ft. lower than the reservoir at its maximum level, causes great volumes of water per linear foot to flow over the dam. The drop from the crest to the concrete apron at the foot of the dam is 59 ft., and the thickness of the apron where the water falls upon it is about 12 ft. Projecting above the apron, close to the foot of the dam, there are two rows of baffles with vertical up-stream faces, those in the upper row being about 18 ft. wide and 8 ft. high. The baffles in the two rows are staggered, so that the water passing between those in the upper row impinges upon those of the lower row. When in action, much of the water is thrown high into the air, with a nearly complete destruction of its horizontal velocity, and cross-currents are also formed. The total result is that most of the energy of the water is taken care of at the baffles, and beyond them the water flows away with a comparatively low velocity. The baffles are faced on the up-stream side with heavy cast iron anchored to them, and these in turn are securely anchored by reinforcing rods to the mass of concrete beneath.

It seems unnecessary to multiply illustrations. There are many expedients by which the high velocity of the water falling over a dam can be checked very close to its foot, and the dangerous features of the high velocity and the hydraulic jump farther down stream can be avoided.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JOHN WILLIAM EBER, M. Am. Soc. C. E.*

DIED AUGUST 18TH, 1915.

John William Eber was born at Jersey Shore, Pa., on July 14th, 1871. He received his early education at, and was graduated from, the High School at Williamsport, Pa. During the next five years, he studied higher mathematics and engineering with a tutor and independently.

Mr. Eber entered the employ of the West Shore Railroad (New York Central and Hudson River Railroad, Lessee), on February 28th, 1890, serving as Rodman, Transitman, Draftsman, and General Assistant to the Assistant Engineer of the Buffalo Division, at Syracuse, N. Y., until November, 1896, when he was appointed Assistant Roadmaster. He was engaged in that capacity and, later, as Supervisor of Track until 1902, when he assumed a similar position with the New York Central and Hudson River Railroad at Albany, N. Y., having charge of the reconstruction and enlargement of the West Albany yards and many works of a similar nature. In April, 1904, Mr. Eber was appointed Engineer of the Rome, Watertown and Ogdensburg Division, and in the following year Assistant Engineer of Maintenance of Way and Engineer of Track, with jurisdiction over 3 785 miles of road. From 1909 to 1910, he served as Assistant Superintendent of the New York Central Stockyards, at Buffalo, in charge of reconstruction, design, maintenance, and on special work.

In 1910, Mr. Eber was appointed Superintendent of the Adirondack Division of the New York Central between Utica, N. Y., and Montreal, Ont., Canada, in charge of both maintenance and operation. In May, 1912, he left the service of that company to accept the position of General Superintendent of the Toronto, Hamilton and Buffalo Railway, at Hamilton, Ont., Canada. On January 1st, 1913, he was promoted to be General Manager of the latter road.

Owing to ill health, Mr. Eber resigned his position in June, 1915. He died on August 18th, 1915, and was buried at Williamsport, Pa.

He possessed qualities for thoroughness, had fine tact, and, throughout his career, was a favorite with all those with whom he came in contact.

Mr. Eber was elected a Member of the American Society of Civil Engineers on September 4th, 1907.

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* Memoir prepared by F. F. Backus, Esq., Assistant to the President, Toronto, Hamilton and Buffalo Railway Company, Hamilton, Ont., Canada.

ALEXIS HENRY FRENCH, M. Am. Soc. C. E.*

Died May 3d, 1915.

Alexis Henry French was born at North Weymouth, Mass., on May 2d, 1851, and was the son of Henry J. and Lucy H. (White) French.

He received his education in the public schools of his native town, and having been graduated from the Weymouth High School, became an apprentice in the office of Shedd and Sawyer, at that time a well-known engineering firm of Boston, Mass. In 1871, he was appointed an Assistant to Mr. George Tyler, the Engineer and Superintendent of Streets of Brookline, Mass.

Later, Mr. French entered the Massachusetts Institute of Technology as a special student, being associated with the class of 1873. He spent two detached years at the Institute, all the time that he felt he could afford.

On the retirement of Mr. Tyler in 1875, Mr. French was given the use of an office in the Town Hall of Brookline under an arrangement by which he did engineering work for the town and for private parties. This arrangement continued until 1894, when the salaried office of Town Engineer was created, and he was appointed to and held the office until his death.

The title of Town Engineer indicates but slightly the extent and importance of Mr. French's work. As a town, Brookline is unique in having probably the largest population of any municipality retaining the old New England form of town government; and it is also very wealthy, so that its public works are similar to those of a city several times its size. Such a condition requires engineering services of a high order, and this condition Mr. French very ably met.

He saw Brookline grow from a population of 8 000 to one of 30 000. Nearly the entire sewerage and drainage systems of the town, involving a large expenditure and the overcoming of many technical difficulties, were constructed under his direction. The engineering work required in laying out and constructing the Park and Playground System was extensive, and the result is a visible and constantly appreciated monument to the skill with which Mr. French carried out his own designs and those of others. The stone arches across the Park waterway were nearly all designed in his office, and were all built under his direction. Some of these have attracted the attention of

* Memoir prepared by Edward W. Howe, M. Am. Soc. C. E., from information furnished by Henry F. Bryant, Esq., Brookline, Mass.

many engineers, architects, and artists, on account of their great beauty and artistic setting.

The laying out of new streets, the designing and construction of bridges, and the abolition of grade crossings also constituted a large part of the work requiring the Town Engineer's attention. Besides, there were the innumerable minor matters which always make up a large part of the work of the municipal engineer. Mr. French's large experience and wide observation enabled him to meet all these requirements in a way that won him the confidence and respect of all those in any way connected with town affairs, as well as of his contemporaries in the Engineering Profession.

Mr. French had suffered for several months from arterio-sclerosis and had been obliged, in the latter part of 1914, to give up his usual business, except such as he could attend to at his home. His death, which occurred on May 3d, 1915, came quite suddenly, he having celebrated with his family only the day before his sixty-fourth birthday; the occasion was a happy one, and he seemed to be in excellent spirits. He is survived by his wife who was Miss Alice Blanchard Loud, of Weymouth, Mass., whom he married on January 14th, 1880.

Mr. French was a member of the following organizations: The Boston Society of Civil Engineers, of which he was President in 1900; the Massachusetts Highway Association, the Engineers' Club of Boston, the Appalachian Mountain Club, of which he was a Past-President, and several local societies, in all of which he took an active interest. He was an active member of the Harvard Congregational Church, in Brookline, having been for a long time a member of its Prudential Committee.

Mr. French was a great lover of outdoor life, and many of his vacations were spent in the forests and on the mountain tops of New England. Many years ago he took up photography as a recreation, and practised the art with great skill and success. His collection of mountain views presented to the Appalachian Mountain Club have been highly praised for their execution and artistic merit.

Few men have left behind them a more enviable record of integrity and devotion to duty and a willingness to give of themselves for the advancement of mankind. As a public official, his quiet, unassuming methods, unfailing courtesy, and freedom from all political or personal influences tending to divert him from what he felt was his duty to those whom he served, furnish a good example which those called on to serve their fellow citizens in a like capacity may well follow.

Mr. French was elected a Member of the American Society of Civil Engineers on June 6th, 1894.

WILLIAM HUNTER, M. Am. Soc. C. E.*

DIED APRIL 2D, 1915.

William Hunter was born on May 25th, 1854, at Moselem, Berks County, Pa. He was educated at the Polytechnic College of the State of Pennsylvania, from which he was graduated in 1872 with the degree of C. E. He immediately began practising his profession by entering the service of the Philadelphia and Reading Railroad Company, and gradually arose from the position of Rodman to that of Assistant Engineer.

In 1876 he engaged in private practice in the ore mines at Moselem for about a year, when he again took up railroad work by entering the employ of the Pittsburgh and Lake Erie Railroad Company as Division Engineer. He remained with that company until the fall of 1878, when he returned to The Reading as Assistant Engineer. Later, he was advanced to Assistant Road Master, then Assistant Chief Engineer, and on August 9th, 1900, on the retirement of the late Col. H. K. Nichols, he became Chief Engineer, which position he held until his death.

While he was Chief Engineer Mr. Hunter built the Port Reading Railroad, together with the extensive coal docks at Port Reading, the Reading Belt Line Railroad, the Norristown and Main Line Connecting Railroad, the New York Short Line Railroad, the Delaware River Bridge at Yardley, the Rutherford and St. Clair Yards, the Low-Grade Freight Line at Wayne Junction, Philadelphia, and rebuilt the Philadelphia, Harrisburg and Pittsburgh Railroad.

In co-operation with the engineers of the City of Philadelphia, he also executed the work necessary to abolish grade crossings on the Philadelphia, Germantown and Norristown Railroad between Green Street and Wayne Junction, and on the Richmond Branch of the Philadelphia and Reading Railway between Somerset and Richmond Streets.

He was a member of the Engineers' Club of Philadelphia and the Franklin Institute. At the time of his death he was Vice-President of the Philadelphia Association of Members of the American Society of Civil Engineers.

Mr. Hunter was an engineer of marked ability and tireless energy, and held broad and progressive views of the railroad problems of his time. His professional attainments, sound judgment, and constructive skill admirably adapted him to meet the responsibilities of a successful career.

* Memoir prepared by George S. Webster, Edward B. Temple, and Samuel T. Wagner, Members, Am. Soc. C. E., a Committee of the Philadelphia Association of Members of the American Society of Civil Engineers.

As Chief Engineer of a great railroad system, serving many large cities, he had many problems to solve with municipal authorities, and, though he was a man of great firmness of character, he discussed proposed improvements with such an open-minded view of the rights of all parties that he won the esteem and confidence of those who had dealings with him.

His uniform courtesy made it a pleasure to be in his company, and he was possessed of qualities of mind and heart which endeared him to his friends and commanded the respect and admiration of all who came in contact with him either professionally or socially.

Mr. Hunter was elected a Member of the American Society of Civil Engineers on June 5th, 1895.

WILLIAM CORNELL JEWETT, M. Am. Soc. C. E.*

Died May 2d, 1915.

William Cornell Jewett was born in San Francisco, Cal., on December 16th, 1853. He was the son of William Cornell Jewett, of New York, and Almira Guion, the daughter of a prominent citizen of Cincinnati, Ohio. His ancestry, originally of French Huguenot, English, and Dutch extraction, was purely American for more than 200 years.

At the age of sixteen, he was sent from California to Cincinnati to continue his education, and, in due time, he was graduated from the Chickering Institute, at that time among the most noted polytechnic schools in the Ohio Valley.

Soon after his graduation, Mr. Jewett was employed in the office of Mr. S. W. Hartwell, a civil engineer in Cincinnati. In June, 1873, he joined the construction force of the Cincinnati Southern Railroad, beginning as Chainman and advancing by promotion to the position of Clerk and Office Assistant to the late George B. Nicholson, M. Am. Soc. C. E., Division Engineer, continuing in this position until about the end of 1877, except for a few weeks' service as Chainman and Rodman on the surveys for the Covington and Pound Gap Railroad.

During a part of 1881, Mr. Jewett was Resident Engineer's Assistant on the Richmond and Alleghany Railroad. Following this, he was Resident Engineer on the Toledo, Delphos and Burlington Railroad, at Wellston, Ohio, in charge of 13 miles of construction work.

From January, 1882, to August, 1883, he was Resident Engineer on the New Orleans and North Eastern Railroad in charge of 14

* Memoir prepared by S. Whinery, M. Am. Soc. C. E.

miles of construction and 30 miles of track-laying, under Mr. Nicholson, Division Engineer.

After a few months' service as Assistant in the office of the Division Engineer, Northern Division, he was transferred to the general office of the Cincinnati, New Orleans and Texas Pacific Railroad, at Cincinnati, Ohio, where under the direction of the late L. G. F. Bouscaren, M. Am. Soc. C. E., Chief and Consulting Engineer of the System, he made a study of railroad ferry transfers, and designed the transfer ferry across the Mississippi River at Vicksburg, Miss., on the Vicksburg, Shreveport and Pacific Division. In August, 1885, he was advanced to the position of Resident Engineer of the Cincinnati, New Orleans and Texas Pacific Railroad, between Cincinnati, Ohio, and Birmingham, Ala., having charge of lining tunnels, building shops, laying out yards, and other construction work.

In August, 1893, Mr. Jewett was appointed Chief Engineer of the 30-mile extension into Cleveland of the Cleveland, Lorain and Wheeling Railroad, and remained in charge of the location and construction of this important work until its completion.

Returning to Cincinnati in 1897, he was engaged for 12 years on the construction of the new water-works for that city, one of the largest and most important water-supply projects of the time, of which Mr. Bouscaren was Chief Engineer until his death, when he was succeeded by George H. Benzenberg, Past-President, Am. Soc. C. E.

The preliminary and location work on the great settling reservoirs at California, Ohio, was done by Mr. Jewett, after which, as Resident Engineer, he had charge of the construction of these reservoirs and the completion, after the death of Alfred Petry, M. Am. Soc. C. E., of all the work in that vicinity, including the coagulating basins, the construction of the river pumping station and coal-storage buildings, the general supervision of the erection of the four 30 000 000-gal. pumping engines, the laying of the mains connecting these pumps with the settling reservoirs, the intake on the Kentucky shore, and grading the grounds, building roads, and other work. Some idea of the importance of this work may be obtained from the fact that the great settling basins cover 80 acres and have a capacity of 330 000 000 gal.

Of Mr. Jewett's services on this work, an official connected therewith writes:

"In all this work he proved himself to be a valuable assistant, as he was exceedingly conscientious in the performance of every duty assigned to him, and exacted from every Contractor the full performance of all contract requirements, while his reports were always precise and complete."

His last important professional engagement was that of Chief Engineer of Construction of the new Cincinnati General Hospital.

This is one of the largest and most carefully designed and constructed for its purposes among modern hospitals, comprising eighteen large buildings, with every modern improvement that medical science and experience could suggest. The cost was approximately $4 000 000.

On the completion of this work, Mr. Jewett took a well-earned and much-needed vacation, going to his native State, California, where his mother and brothers still lived at Santa Paula, and where, in the midst of beautiful scenery and surroundings, sitting in the shade of a grand live oak, he was attacked by heart failure and expired.

On hearing of his death, the Commissioners of the Cincinnati General Hospital met and adopted the following resolution:

"The sudden death of Wm. C. Jewett on May 2d has brought deep sorrow to this commission whom he represented as Chief Resident Engineer throughout the whole period of construction of the Cincinnati General Hospital.

"In recognition of his splendid and faithful services to the City and this Board be it:

"*Resolved*: That we, the Board of Hospital Commissioners, having by constant and intimate association learned to respect and deeply appreciate the high efficiency, incorruptible and sterling qualities of Mr. Jewett, desire to pay this last tribute to his memory, and to point to him as an excellent example of the ideal public servant. * * *"

This is only one example of the high regard and confidence in which Mr. Jewett was uniformly held by his employers and superiors.

His professional work was characterized by competency, sound judgment, painstaking care and accuracy, energy, and entire devotion to his duties. He was one of those engineers who assumed that contracts and specifications mean what they clearly state, that their requirements are intended to be complied with, and that it is the right and duty of the engineer to enforce them, strictly, but justly. Accustomed, himself, to scrupulous compliance with all his engagements, he expected something of the same fidelity from others.

Personally, he was a man of the highest integrity and honor, modest and quiet-mannered almost to a fault. A constant student, fond of scientific pursuits, he was most at home with his family and his books, but was not neglectful of the duties of the good citizen.

Mr. Jewett was married in 1878, to Ella Gibson, the daughter of John B. Gibson, of Cincinnati, who, with a daughter and son, the latter engaged in engineering, survives him.

Summing up the professional life and work of Mr. Jewett, it may be said that he belonged to that large class of engineers whose work honors the Profession, but who shun rather than court popular or professional credit and distinction, finding their sufficient reward in the consciousness of duties courageously met and ably and faithfully performed.

He had been a member, since 1890, of the Cincinnati Engineers' Club, of which he was a Director in 1898, Vice-President in 1900, and President in 1901. He also belonged to The American Association for the Advancement of Science and the Natural History Society of Cincinnati.

Mr. Jewett was elected a Member of the American Society of Civil Engineers on June 3d, 1885.

HENRY GURNEY MORRIS, M. Am. Soc. C. E.*

Died January 19th, 1915.

Henry Gurney Morris was born in Philadelphia, Pa., on May 25th, 1839.

After his graduation from Haverford College, he entered the manufacturing business and, while quite young, became a member of the celebrated firm of Morris, Tasker and Company, which Company was the first to manufacture wrought-iron pipes and boiler flues.

Mr. Morris afterward became the sole owner of the Southwark Foundry, and constructed some of the largest blowing engines, pumps, and other heavy machinery used at that period, being justly considered at that time as one of Philadelphia's greatest captains of industry.

Early in his business career Mr. Morris became interested in electrical engineering and, with Mr. Pedro G. Salom, one of his intimate business associates, he invented and developed the "Electric Vehicle", for which they received the gold medal of the "Times-Herald Motocycle Contest" in Chicago in 1895, and also the John Scott Legacy Medal of the Franklin Institute. Mr. Morris took out numerous patents in connection with this "vehicle" and also for storage batteries.

He was always a leader in new enterprises and inventions which promised to be of benefit to the country, and was one of the few men in the United States to recognize the importance and value of the Bessemer process of steel manufacture. He was also a leading designer and manufacturer of all kinds of machinery for sugar plantations and refineries, gas plants, and water-works, and early recognized the value of compound engines for use in marine engineering.

Mr. Morris was not quick to make friends, and to those who knew him only casually, he seemed to be somewhat haughty and austere; but to those to whom he gave his friendship, he was gentle, loving. and trusting as a child. He was unusually quiet and reserved in manner, but he had a keen sense of humor and a quick appreciation of merit and ability which kept him greatly in advance of his time.

* Memoir prepared by the Secretary from information supplied by Pedro G. Salom, Esq., Philadelphia, Pa.

His general knowledge of mechanics was extraordinary, and covered nearly every branch of mechanical engineering. It has been said of him that he could build anything from a bicycle to a locomotive and from a motor-boat to a dreadnaught.

Mr. Morris is survived by his widow, who was Miss Sallie Marshall Morris, and three sons.

He was a Member and Vice-President of the American Society of Mechanical Engineers; a Member and President of the Engineers' Club of Philadelphia; a Member of the American Institute of Mining Engineers, the Franklin Institute, and many other National Societies. With a single exception, he was, at the time of his death, the oldest living member of the Union League of Philadelphia, having been elected to membership in February, 1863.

Mr. Morris was elected a Member of the American Society of Civil Engineers on December 4th, 1867, and served as a Director during 1886.

ISAAC RICH, M. Am. Soc. C. E.*

Died March 11th, 1915.

Isaac Rich, son of the late Thomas and Maria L. Rich, of Brookline, Mass., was born there on October 6th, 1856. He prepared at the Boston English High School for the Massachusetts Institute of Technology, where he was a member of the Class of 1878; and the greater part of his useful and honorable life was given to the service, as a civil engineer, of various New England railroads.

His first experience came in college vacation employment as a Rodman on the surveys of the Boston, Hoosac Tunnel, and Western Railroad, between Hoosick Falls and Mechanicville, N. Y. Then, following graduation, his earlier years were divided between New England and the Far West; in 1878-79, on the Atchison, Topeka and Santa Fé, through the Royal Gorge and Grand Canyon of the Arkansas, in Colorado; in 1879-80, on the New York and New England Railroad, constructing the main line from Danbury, Conn., to Towners, N. Y.; during 1880-82, Construction and Resident Engineer on the California Southern Railroad, with supervision over four divisions; in 1882-85, Assistant Engineer for the New York and New England Railroad, with a great variety of work; during 1885-88, Assistant Superintendent, Engineer, and Accountant for the Napa Consolidated Quicksilver Mining Company at Oak Hill, Cal.

* Memoir prepared by J. W. Pearson, Esq., Division Engineer, New York, New Haven and Hartford Railroad Company, Boston, Mass.

In October, 1888, Mr. Rich returned, permanently, to New England, and there was engaged in his profession until illness made his retirement imperative, in May, 1912. From 1888 to 1892, he was with the New York and New England Railroad as Assistant Engineer; from 1892 to 1895, with the Boston and Maine Railroad as Assistant Engineer and Division Engineer; and from April, 1895, until the close of his active years, with the New York, New Haven and Hartford Railroad, as Assistant Engineer on that part of the system which was the original Old Colony Railroad. In this position he performed important tasks in all the varied lines of construction and maintenance work that come to a railroad engineer, and his professional ability, personal integrity, and devotion to duty were appreciated and admired by all who knew him and his work.

Mr. Rich married Miss Alice Montague Vinal, daughter of the late Robert A. and Almira L. Vinal, in 1889, and from that time to his death resided in Somerville, Mass. Besides his widow, he is survived by a brother, Henry Rich, and a sister, Mrs. John A. E. Hussey, both of Brookline.

Devoted to his home and to his work, content in the happiness and the occupation they afforded him, Mr. Rich had little time or inclination for other interests. Quietly genial and kind, unobtrusively helpful, always efficient, Isaac Rich will long be remembered as a good man, a loyal friend, and a capable engineer, who gave the best that was in him, always, to the work that came to his hand.

Mr. Rich became a member of the Boston Society of Civil Engineers in 1890. He was elected a Member of the American Society of Civil Engineers on May 6th, 1903.

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"THE ECONOMICAL TOP WIDTH OF NON-OVERFLOW DAMS." WILLIAM P. CREAGER. (To be presented Jan. 5th, 1916.)

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"The Automatic Volumeter." E. G. HOPSON. (To be presented Dec. 1st, 1915.).Oct., "

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PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

VOL. XLI—No. 10

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

December, 1915

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PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XLI—No. 10

DECEMBER, 1915

Edited by the Secretary, under the direction of the Committee on Publications.

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American Society of Civil Engineers

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The House of the Society is open from 9 a. m. to 10 p. m. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

House of the Society—220 West Fifty-seventh Street, New York.

Telephone Number........................1446 Circle.
Cable Address........................"Ceas, New York."

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

SOCIETY AFFAIRS

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MINUTES OF MEETINGS

OF THE SOCIETY

November 17th, 1915.—The meeting was called to order at 8.30 P. M.; J. Waldo Smith, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 101 members and 12 guests.

A paper by Edwin Duryea, Jr., M. Am. Soc. C. E., and H. L. Haehl, Assoc. M. Am. Soc. C. E., entitled "A Study of the Depth of Annual Evaporation from Lake Conchos, Mexico," was presented by the Secretary, who also read communications on the subject from Messrs. William S. Post, E. F. Chandler, T. K. Mathewson, and John E. Stirling Thorpe.

The Secretary announced the election of the following candidates on November 3d, 1915:

As Members

Howard Clifton Griswold, Chicago, Ill.
Walter Harold Kirkbride, Sacramento, Cal.
Homer Peter Ritter, Washington, D. C.

As Associate Members

Frank Monzon Aguirre, Cienfuegos, Cuba
Charles Lyle Allen, Ronceverte, W. Va.
Everett N Bryan, Modesto, Cal.
Selleck Townsend Mann Carpenter, Buffalo, N. Y.
Harry Seykora Comly, Dunsmuir, Cal.
Quentin Fraser Disher, Woodmere, N. Y.
Joseph Cummings Dort, Salt Lake City, Utah
Rowland Leonard Egenhoff, Oakland, Cal.
Edwin Archibald Fraser, New York City
John deBarth Walbach Gardiner, New York City
Chester Arthur Garner, Warsaw, Ind.
Howard Ray Gass, Jefferson City, Mo.
William Asa Glass, Marathon, Fla.
Eugene Lucius Grunsky, San Francisco, Cal.
John Andrew Hamilton, St. Joseph, Mo.
Earle Underwood Henry, Port Arthur, Tex.
Edward Augustus Clyde Hoge, New York City
Selwyn Simon Jacobs, Chicago, Ill.
Hurlbut Smith Jacoby, Cleveland, Ohio
David Farquharson McCurrach, Seattle, Wash.
Stanley Hastings McMullen, El Paso, Tex.
Rankin York Middleton, Jacksonville, Fla.
William Gerald Moore, Baltimore, Md.
Albert Leo Brecht Moser, Denver, Colo.
Nathaniel Atherton Richards, New York City
Charles Wilson Richey, Pittsburgh, Pa.
George Allen Ridgeway, Columbia, Mo.
William Milo Rumsey, San Diego, Cal.
Walter Samans, Greenville, Pa.
Lewis Norris Whitcraft, Pittsburgh, Pa.

As Juniors

Herschel Heathcote Allen, Towson, Md.
Clifford Walter Anderson, Centralia, Wash.
Futoshi Arakawa, Hilo, Hawaii
Charles Wightman Barber, Washington, D. C.
Ziang Yien Chow, Shanghai, China

George Ledlie Dunlap, Medina, N. Y.
Bertram Kellogg Dunshee, San Luis Obispo, Cal.
William Elton Grafman, Brooklyn, N. Y.
Thomas Carol Higgins, Bar Harbor, Me.
Ernest Leslie Osborne, New York City
James Roy Rosenfeld, New York City
Joseph Nelson Royal, Elmhurst, N. Y.
Samuel Isaiah Sacks, Philadelphia, Pa.
Seth Heness Seelye, Madison, Wis.
Alan Frank Williams, Monrovia, Cal.

The Secretary announced the transfer of the following candidates on November 3d, 1915:

From Junior to Associate Member

James Kip Finch, New York City
John Francis Greathead, White Plains, N. Y.
Hugh Nawn, Roxbury, Mass.
Asa Glisson Proctor, Woodland, Cal.
Maurice Roos Scharff, Pittsburgh, Pa.
Harry Seel Stanton, Elephant Butte, N. Mex.
James Blaine Swickard, Salinas, Cal.
Richard Gaines Tyler, Paris, Tex.

The Secretary announced the transfer of the following candidates on November 10th, 1915:

From Associate Member to Member

Lynn John Bevan, New York City
Harry Alson Briggs, Brown Station, N. Y.
Henry John Brunnier, San Francisco, Cal.
Robert Lemmon Burwell, Baltimore, Md.
John Philip Hogan, Tompkinsville, N. Y.
Jonathan Jones, Philadelphia, Pa.
George Kirkpatrick Larrison, Honolulu, Hawaii
Thomas Elmer Phipps, Seattle, Wash.
Carl Howell Reeves, Seattle, Wash.
Waldemar Spaulding Richmond, Detroit, Mich.
Charles Bruce Scott, Richmond, Va.
Harry Ives Shoemaker, Manila, Philippine Islands
Frank Henry Stephenson, Cleveland, Ohio

The Secretary announced the following deaths:

Charles Chancellor Wentworth, of Roanoke, Va., elected Member, April 4th, 1888; died November 11th, 1915.

William Frederick Allen, of New York City, elected Associate Member, January 3d, 1900; died November 9th, 1915.

Adjourned.

December 1st, 1915.—The meeting was called to order at 8.30 P. M.; Robert Ridgway, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 141 members and 17 guests.

The minutes of the meetings of October 20th and November 3d, 1915, were approved as printed in *Proceedings* for November, 1915.

A paper by E. G. Hopson, M. Am. Soc. C. E., entitled "The Automatic Volumeter", was presented by the Secretary.

A paper by Clement E. Chase, Jun. Am. Soc. C. E., entitled "The Cherry Street Bridge, Toledo, Ohio", was presented by the Secretary, who also read communications on the subject from Messrs. Edward Godfrey and James Ritchie. The paper was discussed orally by Messrs. T. Kennard Thomson and Max M. Miller.

The Secretary announced the following death:

JOEL HERBERT SHEDD, of Woonsocket, R. I., elected Member, September 15th, 1869; died November 27th, 1915.

Adjourned.

December 15th, 1915.—Because of the necessity of going to press with this number of *Proceedings* in advance of this meeting, the publication of its minutes must be deferred until January, 1916.

OF THE BOARD OF DIRECTION

(Abstract)

November 9th, 1915.—The Board met at 10 A. M.; President Marx in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bontecou, Bush, Coleman, Crocker, Davies, Edwards, Fuller, Greiner, Harwood, Haskell, Hawley, Herschel, Hodge, Keefer, Leonard, Loweth, McDonald, Ockerson, Swain, Tuttle, and Williams.

A Report of a Committee of the Board on Re-districting of the Society consisting of Messrs. Davies, Tuttle, and Hunt, was presented.*

On motion, duly seconded, the Report was received and all its recommendations adopted, and its publication in *Proceedings* for the information of the membership ordered.

A Committee was appointed to consider the question of retrenchment in Society expenditures, and to report at the next meeting of the Board.

The following resolution was adopted:

"That a Committee of the Board be appointed to supervise the work of all Special Committees, to recommend appropriations, and pass upon all disbursements under the general direction of the Board."

A Report from Messrs. A. S. Tuttle, H. W. Hodge, and A. D. Flinn, appointed to represent the Society at the Constitutional Convention of the State of New York, was presented.†

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* See page 736.

† See page 741.

On motion, duly seconded, this Report was ordered published in *Proceedings* for the information of all members, and the recommendation of the Committee that a similar committee be appointed to continue the work was adopted as the action of the Board.

A Report of the Tellers who canvassed the Preliminary Suggestions for Nominating Committee in the various Districts was presented.

The time and place for holding the Annual Convention of 1916 was considered, and it was decided that the Convention should be held in Pittsburgh, Pa., the time, if possible, to be fixed as the last week in June.

The President was authorized to appoint a Local Committee of Arrangements for the Annual Meeting.

President Charles D. Marx was appointed to represent this Society on the John Fritz Medal Board of Award for the next four years, to take the place of Past-President Mordecai T. Endicott whose term of office will expire in January.

A Committee was appointed to draft the Annual Report of the Board of Direction.

Special Meetings of the Society for the consideration and discussion of the Report of the Special Committee on Materials for Road Construction to be held on Friday, January 21st, 1916, were authorized.

The resignations of 3 Members, 2 Associate Members, and 3 Juniors, were accepted, to take effect December 31st, 1915.

Adjourned.

November 10th, 1915 (Adjourned Meeting).—The Board reconvened November 10th, 1915; President Marx in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bontecou, Bush, Coleman, Crocker, Harwood, Herschel, McDonald, Tuttle, and Williams.

Thirteen Associate Members were transferred to the grade of Member.

Applications were considered and other routine business was transacted.

Adjourned.

REPORT IN FULL OF THE FIRST AND SECOND SESSIONS (THE SECOND SESSION BEING THE BUSINESS MEETING) OF THE FORTY-SEVENTH ANNUAL CONVENTION, SAN FRANCISCO, CAL.

FIRST SESSION

Thursday, September 16th, 1915.—The first session of the Convention was opened in the St. Francis Hotel at 10 A. M.; Charles D. Marx, President, Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, about 400 members and guests.

THE PRESIDENT.—I will now call the Forty-seventh Annual Convention of the American Society of Civil Engineers to order.

Some two weeks ago or so a Mr. Estabrook from New York delivered a very eloquent address on the subject of "What is a Constitution among Friends". He called particular attention to the fact that if in the Constitution of the United States there are some things which are objectionable there are legal ways for bringing about the necessary changes. He went on and illustrated by citing a number of things that had been changed in this way. Now, after listening to him, I, of course, too, have become a very strict constitutionalist; and as I looked over the Constitution of our Society I realized that there were certain things that called for a change. Among those things is the paragraph which requires the President on this occasion to deliver an address; but, as I told you, there are legal ways for bringing about a change if anything does not suit you, and a Committee of the Society has been appointed to make such changes, when the membership approves, as seem desirable. If as a result of my talk to you to-day a change should be made which will prevent future presidents from inflicting an address on you, I think my efforts will not have been in vain. (Applause.)

In view of the fact that there is to be next week a meeting, international in character, entitled "International Congress of Engineering", and that before that Congress there will be presented a large number of papers which summarize the present status of engineering, not only in this country but all over, I have felt free to depart somewhat from the general character of presidential addresses and to take up a subject which for many years has appealed to me. We are supposed to be materialists, but I have always felt that if there is any profession which tends to develop the ideal in men it is the Profession of Engineering. I have therefore chosen as the subject of my talk to-day "Idealism and Art in Engineering". I hope you will bear with me for a little while.

(The President then delivered the Annual Address.*)

* The President's Address is printed in *Proceedings* for October, 1915, on pages 2071 to 2086 of Papers and Discussions; and also in *Transactions*, Vol. LXXIX, on pages 1329 to 1344.

SECOND SESSION—BUSINESS MEETING

Thursday, September 16th, 1915.—Immediately after the President had delivered the Annual Address, the Business Meeting was called to order.

The President.—Mr. Secretary, I think you have some announcements to make. The meeting will now be called to order as the Business Meeting of the Society.

The Secretary.—Mr. President, the suggestions as to the place for holding the Annual Convention of 1916 and the time for holding it have come in as follows:

Pittsburgh, Pa	122	Detroit, Mich	10
Boston, Mass	61	Kansas City, Mo	10
Chicago, Ill	49	St. Louis, Mo	10
Washington, D. C	15	Minneapolis, Minn	9
Atlantic City, N. J	13	Saratoga, N. Y	7
New York City	13	Galveston, Tex	6
New Orleans, La	12	Duluth, Minn	5
Philadelphia, Pa	11	Great Lakes	5
Denver, Colo	11		

The following have received 4 votes each: Atlanta, Ga., Cincinnati, Ohio, Cleveland, Ohio, Jacksonville, Fla., Portland, Ore., Salt Lake City, Utah.

The following have received 3 votes each: Birmingham, Ala., Buffalo, N. Y., Dallas, Tex., Los Angeles, Cal., Niagara Falls, N. Y., Quebec, Que., Canada, Richmond, Va., St. Paul, Minn.

The following have received 2 votes each: Cape May, N. J., Dayton, Ohio, French Lick Springs, Ind., Havana, Cuba, Mackinac Island, Mich., Nashville, Tenn., Omaha, Nebr., Providence, R. I., Rochester, N. Y., Seattle, Wash., Vicksburg, Miss.

The following have each received 1 vote: Asheville, N. C., Baltimore, Md., Bermuda, Bretton Woods, N. H., Butte, Mont., Catskill Mountain House, N. Y., Columbus, Ohio, El Paso, Tex., Fresno, Cal., Hot Springs, Ark., Houston, Tex., Honolulu, Hawaii, Louisville, Ky., Manchester, Vt., Montreal, Que., Canada, New London, Conn., Portland, Me., Portsmouth, N. H., San Antonio, Tex., Syracuse, N. Y., Thousand Islands, N. Y., Utica, N. Y., Waukesha, Wis.

Suggestions as to the Time for holding the Annual Convention of 1916 have been received, as follows:

January	4	July	21
February	2	August	13
March	2	September	103
April	3	October	34
May	12	November	2
June	96	December	4

April or May	2
May 25th or September 2d.	1
May or June	3
May or September	3
May or October	2
June or July	5
June or September	5
June, July or August	1
Between June and September 15th	1
June to October	1
July or August	2
July, August, or September	1
August or September	2
September or October	2
September or November	1
October or November	1
December, January, or February	1
Early Spring	1
Spring	1
Summer	2
Fall	1
Late Fall	1
Winter	1
Spring, during Session of Congress. (This time is suggested in connection with Washington, D. C., as the place.)	1
Last week in June "Race Week". (This time is suggested in connection with New London, Conn., as the place.)	1
During Texas State Fair. Last half of October. (This time is suggested in connection with Dallas, Tex., as the place.)	1

THE SECRETARY.—There are also invitations from the following: The Chambers of Commerce of Boston, Mass., Providence, R. I., and Philadelphia, Pa.; and letters from M. O. Leighton, M. Am. Soc. C. E., suggesting Washington, D. C., Burtis S. Brown, Assoc. M. Am. Soc. C. E., suggesting Boston, Mass., and B. F. Groat, M. Am. Soc. C. E., suggesting Pittsburgh, Pa. I received this morning the following telegram:

"PITTSBURGH, PA.,
"September 15, 1915.

"MR. CHAS. W. HUNT, Secretary,
"American Society of Civil Engineers,
"St. Francis Hotel, San Francisco, Cal.

"The American Society of Civil Engineers is cordially invited and urged to select Pittsburgh as the place of meeting for the next Convention. Many places of engineering interest, ample hotel accommodations, golf links, and an absence of forty years justify our hope for favorable action.

"PITTSBURGH COMMITTEE: GEO. S. DAVISON, *Chairman*, ROBERT A. CUMMINGS, *Secretary*, H. M. WILSON, B. MCKEEN, D. W. MCNAUGHER, A. R. RAYMER, R. KHUEN, M. KNOWLES."

I do not think, Mr. President, it is necessary to read the many invitations from various Chambers of Commerce, Mayors of all the towns; it will take up so much time.

THE PRESIDENT.—What is the suggestion, then?

A MEMBER.—Mr. President, I move that the place and time of holding the next Annual Convention be referred to the Board of Direction with power to act.

(The motion was formally presented, seconded, and carried.)

THE SECRETARY.—I want to report, for the information of the members of the Society, that, on the ballot which was issued some time ago on the question of rescinding a resolution which was adopted at one of our Annual Meetings with regard to the licensing of engineers, the ballot was canvassed on August 31st, and that the result was 2 024 ballots in favor of rescinding the resolution, and 592 against; so that the resolution was rescinded in accordance with the recommendation of the Board of Direction.

THE PRESIDENT.—That will now leave the Board of Direction free to act as the occasion may arise.

THE SECRETARY.—Mr. President, at the meeting of the Seattle Association of Members of the American Society of Civil Engineers, held on June 28th, 1915, the following report was received from a special committee, consisting of Messrs. Bertram D. Dean and Robert Howes, appointed to review the paper entitled "Suggested Changes and Extension of the United States Weather Bureau Service in California", and the recommendations therein made were adopted by that Association:

"The paper by Messrs. Binckley and Lee in the February issue of the *Proceedings* of the Am. Soc. C. E., 1915, appears to consider the United States Weather Bureau as having for its primary object the collection of information relative to water supply, and makes sundry criticisms and recommendations.

"Your committee finds that, at least in our district, the Weather Bureau has a large sphere of usefulness in other directions, which must be taken into consideration in locating many of their stations, and we hesitate to criticize the location of existing stations. We are, however, impressed that the development of the State would be materially assisted by extending the observations to supply additional information regarding precipitation, evaporation, and snow storage, in the mountainous territory, which is the chief source of our water supply.

"We recommend that this Association request the parent Society to take definite steps to encourage the establishment of additional stations, and collect information relative to precipitation, evaporation, and snow storage in the mountains of Washington and Oregon as well as California; also to recommend that provision be made for more frequent inspection of stations by the Department's trained field observers."

As that calls for a presentation of the resolution to the Society, sir, I have brought it up.

THE PRESIDENT.—What is your pleasure in regard to this resolution? Do you wish to refer it to the Board? I will entertain a motion.

A MEMBER.—I move it be referred to the Board of Direction.

(The motion being duly seconded, was formally presented and carried.)

THE PRESIDENT.—The motion is carried. It is referred to the Board of Direction.

THE SECRETARY.—I have no further business to bring before the meeting, except that, as the attendance is quite large, I think we ought to call upon the representative of the Local Committee, perhaps Mr. Schneider, to make any announcements as to any change or addition to the programme which has already been distributed.

THE PRESIDENT.—Mr. Schneider will kindly step forward.

E. J. SCHNEIDER, M. AM. SOC. C. E.—The Committee on Arrangements has followed the programme, information regarding which was sent out in a circular by the Society about July 1st. That information has been amplified by papers and data which can be obtained at the Secretary's office, and also at the registration office, both upstairs on the mezzanine floor. However, for those who have not yet read that information, or who are not posted, I will say briefly that the programme is about as follows:

This afternoon we are expected to meet at the Fillmore Street entrance of the Exposition, just inside the grounds, and at 4 P. M. we will be escorted to the Court of Abundance, where the formal ceremony will take place at about 4.30 P. M., and the Civil Engineers will be presented with a bronze plaque.

Between 6 and 7 P. M. there will be an informal reception at the Old Faithful Inn, just within the grounds, at the Van Ness Avenue Entrance; and at 7 P. M., or shortly thereafter, there will be a dinner, at which the members of the Society, who are not members of the local San Francisco Association, are to be the guests of that Association—they and their ladies. This is to be an informal affair, and we trust that every one who can will come. It is very important that you let the committee know as soon as possible—most all of you have, but if there are any more, we should know, and that information should be given at the office, on the mezzanine floor.

On Friday at 2.55 P. M. we have arranged for a special train to go to Del Monte and to stay there over Saturday, and on Sunday take the trip through the Big Basin. This, we think, will be a very attractive programme, and we trust that every one who possibly can will take part.

For those who intend to take part in most of the activities, we have arranged a coupon book, which not only gives admission to a number of the events on the programme, but also gives general information as to what the programme will be. These can be obtained upstairs on the mezzanine floor, and we would like very much for every one who contemplates taking this trip to let us know, so that we can make proper arrangements.

The trip on Sunday from Santa Cruz to San José is to be taken by automobiles, and as these have to be provided for, it is necessary that we know a day or two in advance. I trust that as many as possible will participate. I thank you.

You can have your picture taken immediately after adjournment just across the street in Union Square.

There is just one other point, too: That special train that leaves here at 2.55 P. M. to-morrow will stop at Palo Alto, so that those who want to visit Stanford University can get on the train there instead of coming back to San Francisco.

THE SECRETARY.—Mr. President, I have received this morning invitations for the members of the Society to visit the exhibits of the Bureau of Mines, the American Mine Safety Association, the California Metal Producers Association, the Bureau of Standards, and the United States Steel Corporation.

THE PRESIDENT.—Is there any new business to come before the meeting? Any suggestions?

THE SECRETARY.—I am instructed by the President to ask the members of the Board of Direction to remain in this room after this meeting adjourns. The second thing which the Constitution says must be done at the Annual Convention, the first being the President's address, is the meeting of the Board of Direction.

THE PRESIDENT.—If there is nothing further to come before the meeting, a motion to adjourn is in order.

(On motion made and seconded, the Convention then adjourned.)

MEETINGS AND EXCURSIONS IN CONNECTION WITH THE FORTY-SEVENTH ANNUAL CONVENTION AND THE INTERNATIONAL ENGINEERING CONGRESS, 1915

The Forty-Seventh Annual Convention of the Society was held in connection with the International Engineering Congress, 1915, in San Francisco.

The dates covered by the Convention were September 16th, 17th, and 18th. The Congress was held from September 20th to 25th, inclusive.

Similar meetings of the American Institute of Mining Engineers, the American Society of Mechanical Engineers, and the American Institute of Electrical Engineers, were held on the same dates.

A special train for the accommodation of the members of all these organizations, as well as other members of the Congress, all arrangements for which were in the hands of Mr. Charles Warren Hunt, Secretary of the Joint Committee of the Congress on Entertainment and Transportation, left New York on Thursday, September 9th, and arrived in San Francisco on the evening of Wednesday, September 15th. More than 160 persons, about equally divided between the membership of the Associated Societies, together with a number of foreign participants, made the trip. Stops were made at Niagara Falls for 4 hours, at Colorado Springs for 12 hours, and at the Grand Canyon for more than 12 hours.

The headquarters of the Society were at the St. Francis Hotel.

The Local Committee of Arrangements, Messrs. C. E. Grunsky, H. L. Haehl, Fred R. Muhs, E. J. Schneider, and E. T. Thurston, Jr., had arranged a very delightful programme, which was carried out as follows:

Thursday, September 16th, 10 A. M.—The Convention was called to order at the St. Francis Hotel. President Charles D. Marx in the Chair; Secretary Chas. Warren Hunt; and present, also, about 400 members and guests. The President delivered the Annual Address on "Idealism and Art in Engineering". The time and place for holding the next Convention was considered and other business matters transacted.

At 4 P. M., members of the Society and their guests met at the Fillmore Street entrance of the Exposition and were escorted by Exposition Officers to the Court of Abundance, where, before a gathering of about 500 engineers and guests, a bronze plaque inscribed on one side "American Society of Civil Engineers, September 16th, 1915", and

on the other "In Commemoration Panama-Pacific International Exposition, San Francisco", was presented with appropriate ceremonies by the management of the Exposition. At 7.30 P. M. a most enjoyable banquet was held at the "Old Faithful Inn", in the Exposition grounds, to which all visiting members of the Society and their families were invited by the San Francisco Association of Members. A number of members of other societies were also present, the total number in attendance at this banquet being about 400. The Exposition orchestra, of about 90 pieces, furnished music during dinner, and later there was informal dancing.

Friday, September 17th.—The morning of this day was left open for visits to the Exposition.

Afternoon, 2.55 P. M.—A special train left San Francisco for Del Monte, where the party arrived at 7.30 P. M. Ninety members and guests went on this trip.

Saturday, September 18th.—Members interested in the game of golf played a one-day tournament, having rounds both in the morning and in the afternoon. This enabled them to take a 17-mile automobile drive and to participate in a luncheon at "Pebble Beach Lodge", joining the remainder of the party there. After luncheon the return to Del Monte was made by another route.

Sunday, September 19th.—At 8.30 A. M. the party left Del Monte by special train, arriving at Santa Cruz at 10.25 A. M., and drove from that point by automobile to the Big Basin and California Redwood Park. Luncheon was served at Governor's Camp, after which other points of interest were visited, notably the big trees, some 60 ft. in circumference at the base, and as much as 300 ft. in height. In the afternoon the drive was continued *via* the new State road over Saratoga Summit, thence by county road descending to the Santa Clara Valley and to San José, where dinner for the entire party was served at the Hotel Vendôme. This beautiful mountain drive covered more than 66 miles, and the method used by the Committee in making it interesting to those who participated was very successful. By running over the road previously with a speedometer, 97 points of interest were noted, and pasteboard signs showing the number of miles and tenths of miles printed for each. These were planted along the road by a pilot machine, and the programme of the drive, which was distributed to each person in the party, contained a brief statement of the interesting things to look for at each of these special points. Therefore every one was fully advised in advance of what there was to see at the next sign post, and was on the lookout for it. The party left San José at 9 P. M. and reached San Francisco that evening.

International Engineering Congress, 1915

During the week before the opening of the Congress, headquarters for registration and for the general accommodation of members were established. During the two weeks that headquarters were maintained 851 persons registered. During the week before the Congress, a number of excursions were made by its members to the San Francisco High Pressure Fire System, Gas and Electric Plants, Spring Valley Water Company, the Delta Lands of the Sacramento and San Joaquin Rivers, Great Western Power Company's Hydro-electric Plant at Las Plumas, Lake Spaulding and Drum Power House, Gold Mines at Grass Valley, and Oil Fields at Coalinga. All the meetings of the Congress were held at the Civic Center Auditorium, a building erected by the Exposition for the free use of Congresses and Conventions during the Exposition year.

The opening session was held on Monday at 10 A. M., there being present about 50 official delegates and approximately 800 members and visitors. W. F. Durand, Chairman, Committee of Management, called the meeting to order, and, after a brief address reviewing the origin and development of the Congress, introduced Major-General George W. Goethals, who took the Chair as Honorary President. Addresses of welcome were made by Mayor James Rolph, Jr., of San Francisco, and C. C. Moore, President, Panama-Pacific International Exposition. President Goethals then delivered an address. Speeches were also made by the following representatives of other countries:

Major J. L. de Pulligny, of France.
Senor Don José R. Villalon, of Cuba.
J. B. Challies, of Canada.
Admiral Wei-Han, of China.
Senor Don Fernando Cruz, of Guatemala.
Admiral M. Kondo, of Japan.
H. J. E. Wenckebach, of The Netherlands.
Senor Don Alejandro Canton, of Nicaragua.
Senor Don J. M. de Lasarte, of Spain.
Richard Bernstrom, of Sweden.
Professor Arthur Rohn, of Switzerland.

In the afternoon, a meeting was held before which papers on the Panama Canal were presented. General Goethals presided, and about 800 members and visitors were present.

On the succeeding four days 51 technical sessions were held, as follows: Waterways 4, Irrigation 4, Municipal 5, Railways 5, Materials of Road Construction 5, Mechanical 6, Electrical 3, Mining 3, Metallurgy 5, Naval Architecture 7, Miscellaneous 4. The Chairmen who

presided over these sectional meetings were General Goethals, Messrs. Charles D. Marx, C. McD. Townsend, Elwood Mead, J. L. de Pulligny, P. M. Norboe, George W. Fuller, M. M. O'Shaughnessy, Charles H. Hyde, George H. Pegram, Charles S. Churchill, Charles F. Loweth, Arnold Stucki, William Hood, Mansfield Merriman, William H. Shockley, George W. Dickie, Calvin W. Rice, W. R. Eckart, Jr., W. L. Saunders, J. J. Carty, J. Franklin Stevens, H. J. Ryan, J. W. Richards, L. D. Ricketts, Charles Butters, E. B. Braden, W. L. Capps, W. F. Durand, and Thomas O. Vilter.

The closing session of the Congress was held at 11:30 A. M., Saturday, September 25th. W. F. Durand was Chairman; W. A. Cattell, Secretary; and present, also, about 30 delegates and 500 members and visitors. Addresses were made by Charles D. Marx, President, Am. Soc. C. E., J. A. Brashear, President, Am. Soc. of Mech. Engrs., J. J. Carty, President, Am. Inst. of Elec. Engrs., Maj. Jean L. de Pulligny, representing the Republic of France, J. G. Sullivan, Chief Engineer of the Canadian Pacific Railway, Admiral Wei-Han, of China, Senor Don Fernando Cruz, of Guatemala, Admiral M. Kondo, of Japan, H. J. E. Wenckebach, of The Netherlands, Senor Alejandro Canton, of Nicaragua, Senor Don J. M. de Lasarte, of Spain, and Senor Don José R. Villalon, of Cuba.

During the Congress a number of social functions and excursions were held, as follows:

On Monday evening there was a general reception.

On Tuesday 75 members and guests made an excursion to Mt. Tamalpais. Special visits were made to the Exposition.

On Thursday, September 23d, a reception and lawn party, at which about 500 members and guests were present, was held in "Faculty Glade" at the University of California.

On Friday, September 24th, 76 members and guests, mostly ladies, made an automobile trip to points of interest in the city.

On the evening of Friday, September 24th, 266 members and guests sat down to dinner in the Ball Room of the Palace Hotel. Professor Durand acted as Toastmaster, and speeches were made by Major Jean L. de Pulligny, Brig.-General W. L. Sibert, Professor Charles D. Marx, President, Am. Soc. C. E., representing the five supporting Societies, Chester H. Rowell, representing the Governor of California, M. M. O'Shaughnessy, City Engineer, representing the City of San Francisco, Henry T. Scott, representing the Board of Directors of the Panama-Pacific International Exposition, and Benjamin Ide Wheeler, representing the University of California.

The last event of the Congress before the general closing session was the trip by steamer around San Francisco Bay, by the courtesy of the Board of State Harbor Commissioners.

A list of the papers to be published by the Congress was printed in *Proceedings* for November, 1915, page 654.*

Engineers' Day Ceremonies, San Francisco, Cal.

One of the interesting occasions during the Congress was a meeting held on Friday, September 24th, called by the Directors of the Exposition for the purpose of honoring the Engineers who participated in the Construction and Administration of the Exposition. The ceremonies were arranged at this particular time because the presence of so many Engineers at the Congress made it possible for many members of the Profession to be present.

The meeting was held in the "Court of Abundance". Mr. William H. Crocker, Chairman of the Building and Grounds Committee of the Exposition, presiding, music was furnished by the Philippine Band, and there was an attendance of about 450.

On the platform were seated the following Engineers of the Exposition: L. F. Leurey, Assistant Chief Mechanical and Electrical Engineer, H. D. Dewell, Chief Structural Engineer, Shirley Baker, Construction Engineer, William Waters, Superintendent of Construction, Guy L. Bayley, Chief Mechanical and Electrical Engineer, A. H. Markwart, Assistant Director of Works and Chief of Construction, and H. D. H. Connick, Director of Works. Others present were: C. C. Moore, President of the Exposition, J. A. Britton, of the Board of Directors, Charles D. Marx, President, American Society of Civil Engineers, J. A. Brashear, President, American Society of Mechanical Engineers, J. J. Carty, President, American Institute of Electrical Engineers, C. A. Vogelsang, Commissioner of the Exposition, Charles Warren Hunt, Secretary, American Society of Civil Engineers, Calvin W. Rice, Secretary, American Society of Mechanical Engineers, F. L. Hutchinson, Secretary, American Institute of Electrical Engineers, Lieutenant-Commander C. H. Woodward, U. S. N., Naval Aid to the President, and Howard Holmes, Consulting Engineer, Wharves and Docks, of the Exposition.

Messrs. E. E. Carpenter, Chief Civil Engineer of the Exposition, and W. H. Johnson, Engineer of Fire Protection, who were also to be honored, were unable to be present.

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* The general fee for membership in the Congress ($5.00) covers the index volume and one additional volume. Other volumes may be purchased on a sliding scale of prices: one extra volume costing $3.50 in paper covers ($3.75 in cloth), two volumes, $6.75 in paper ($7.25 in cloth), etc.

Applications for membership in the Congress will be received until the latter part of December, 1915, and should be addressed to William A. Cattell, Secretary International Engineering Congress, 1915, Foxcroft Bldg., San Francisco, Cal., who will gladly answer further inquiries.—(*Secretary.*)

Mr. William H. Crocker spoke as follows:

"LADIES AND GENTLEMEN: We are assembled here to-day, on this perfect day, in this beautiful place, for the purpose of honoring nine of the engineers who worked faithfully in the construction of what you have seen on all sides. The Exposition officials wish to pay them special recognition and they have chosen the time of the meeting of the Engineers' Congress in order to honor these nine engineers who worked for the Exposition, in a more permanent manner than otherwise would be adopted. The work which these gentlemen did shall never be forgotten. I wish to emphasize the fact that they had a great responsibility. It was due to their careful work that we were able to adjust our finances, adjust the work that was to be done, and so to handle the construction of the work as to make it possible under their direction. They did their work faithfully and well. They did their work within the appropriation, and THEY ACCOMPLISHED IT ON TIME. I cannot emphasize too greatly the importance of the services of these gentlemen, and in order to perpetuate that—in order to show them the gratitude the Exposition officials feel toward them jointly and severally —President Charles C. Moore will present to each one of them a token of this occasion."

Mr. Charles C. Moore, President, Panama-Pacific International Exposition, spoke as follows:

"EXPOSITION ENGINEERS, LADIES AND GENTLEMEN: Some have come here from curiosity, some on account of friendship to the Exposition Engineers, and some have come for the more significant—the most significant I should say—feature of to-day's ceremonies, THE RECOGNITION OF THE ENGINEERING PROFESSION. Some have wondered why all these days have gone by and we have waited until now to honor these men. The reason is plain and, I am sure, convincing: There is in session now the great International Engineering Congress. There have been in session here—and many of the members are still here—the great American Societies of Engineering: Civil, Mechanical, Electrical, Mining, Naval, and their allied interests. We thought it wise to defer this day until the engineers were here, from our country and from afar, that they might join with us in the honor we seek to do the Profession by honoring the men who have done such signal service. That is our reason. Those good men standing there are pillars in the structure, and theirs is the glory of this work, and therefore you can understand to some extent why the Exposition management feels a sense of gratitude and a desire to publicly honor the splendid men who have been with us and who have been an honor to their Profession. To-day it seems to me that words are not exactly appropriate. We should be judged by our acts and not by any verbal extravagances. It is intended that in the history of the Exposition, in commanding place, shall be put the names of the engineers who are here to-day, and coupled with that, there will be given, in commanding place too, the obligation that the Exposition is under to the professions of which they are members. Now, there is a great deal that might be said, but I repeat that I want you to judge by the effect of our actions, and your presence, our earnest desire and yours to honor these splendid men who have

done so much for us, therefore, I will ask to come forward, in turn, the engineers named, to receive this token. I will read one. The others are inscribed the same:

'IN RECOGNITION OF
SERVICES AS ONE OF THE ENGINEERS OF THE
PANAMA PACIFIC INTERNATIONAL EXPOSITION
BY ITS BOARD OF DIRECTORS
ON EXPOSITION ENGINEERS' DAY,
SEPTEMBER 24TH, 1915.'

"If it were in our power we would gladly stud its cover and contents with jewels or other marks of great value, but this simple testimonial means, I am sure, to you what it means to us, a recognition of splendid work enthusiastically, conscientiously, and most efficiently done, and I hope that in the records of the Societies now in session here, mention will be made of those who have honored you all by serving us. They are our friends and yours.

"I will ask to come forward first: Assistant Director of Works, A. H. Markwart; Chief Mechanical and Electrical Engineer, G. L. Bayley; Superintendent of Construction, William Waters; Construction Engineer, Shirley Baker; Chief Structural Engineer, H. D. Dewell; Chief Civil Engineer, E. E. Carpenter, having finished his work with the Exposition is absent on an important commission. Mr. Baker will receive his token for him. Engineer of Fire Protection, W. M. Johnson, is likewise absent. Mr. Bayley will receive his token. Assistant Chief Mechanical and Electrical Engineer, L. F. Leurey."

(The Engineers came forward as their names were called, and were presented with their plaques. When they were again seated, Mr. Moore continued):

"Gentlemen, there is one left. One who has been a leader. One who will speak for them all. I do not want to embarrass him any more than embarrassing the other individuals, by personal reference. There is much we would like to say, but we will let this token and our good wishes, our appreciation and affection always, stand for words. I will ask Director of Works, Mr. H. D. H. Connick, to step forward."

The following is Mr. Connick's reply:

"Mr. President, Ladies and Gentlemen: It is particularly pleasant to be an engineer to-day and feel that the Profession is being recognized. Really, the reason that it is being recognized is largely due to these men that you see here. The best trick that I did for this Exposition, by far, was to participate in their selection for the various jobs, because I am quite sure that if I had not been so lucky in picking out such men to fill the heads of departments we would not have gotten along as well as people suspected we did.

"This has been rather a pleasant task in many ways, full of interest —and a particularly gratifying thing has been that we felt, all along, that we were advancing the cause of the engineer. This is really the first American International Exposition where engineers have played such an important part. At other expositions they selected men from other professions, but here engineers have had rather a lot to do, both

with the construction and also the operation, and our whole experience with the Exposition has been an extremely active one. We got along remarkably well, both with the men who have been over us and those who have assisted us in carrying out the work. I thank you."

A brief address was also made by John A. Brashear, President, American Society of Mechanical Enginneers.

Attendance

The following 307 members of the Society were in attendance at the Convention. There were also present 231 ladies and others of the families of members.

Ahern, Jeremiah......Dixon, Cal.
Albert, D. W.....Yuba City, Cal.
Alden, L. T...San Francisco, Cal.
Allan, T. J........Oakland, Cal.
Alvarez, A. C......Berkeley, Cal.
Amweg, F. J..San Francisco, Cal.
Anderson, G. G..Los Angeles, Cal.
Angwin, H. R......Oakland, Cal.
Armstrong, H. A. Sacramento, Cal.
Ash, Dorsey........Berkeley, Cal.

Babcock, W. S....New York City
Bailhache, J. G., San Francisco, Cal.
Baker, A. R.....San Rafael, Cal.
Barnard, A. F..Los Angeles, Cal.
Barnard, W. K..Los Angeles, Cal.
Baxter, F. E. Salt Lake City, Utah
Bayley, E. A....Los Angeles, Cal.
Belknap, W. E....Yonkers, N. Y.
Bensel, J. A......New York City
Bernegau, C. M...New York City
Bienenfeld, A. M., San Francisco, Cal.
Bienenfeld, Bernard, San Francisco, Cal.
Binkley, G. H......Oakland, Cal.
Bixby, F. L.. State College, N. Mex.
Boggs, E. M.......Oakland, Cal.
Bovyer, W. B.. San Francisco, Cal.
Bowers, N. A..San Francisco, Cal.
Brackenridge, W. A., Los Angeles, Cal.
Bradley, W. L......Needles, Cal.
Bright, J. S. San Bernardino, Cal.
Brinkley, M. H. San Francisco, Cal.
Brown, J. M..Vermilion, S. Dak.
Brown, Le Grand, San Francisco, Cal.
Brua, E. G....San Francisco, Cal.
Brunnier, H. J. San Francisco, Cal.
Bumsted, E. B. San Francisco, Cal.
Burrage, J. O. San Francisco, Cal.
Bush, P. L....San Francisco, Cal.
Byers, A. M. C. San Francisco, Cal.
Byers, C. H...San Francisco, Cal.

Cahill, J. R...San Francisco, Cal.
Campbell, H. A., San Francisco, Cal.
Carpenter, J. C., San Francisco, Cal.
Carpenter, S. E....Berkeley, Cal.
Carroll, Eugene......Butte, Mont.
Cattell, W. A. San Francisco, Cal.
Charnley, Walter, São Paulo, Brazil
Christensen, G. A., San Francisco, Cal.
Church, H. R......Berkeley, Cal.
Churchill, C. S.....Roanoke, Va.
Clapp, W. A..Fort McDowell, Cal.
Clark, H. F...San Francisco, Cal.
Clarke, D. D.......Portland, Ore.
Cleary, A. J..San Francisco, Cal.
Collins, A. L..San Francisco, Cal.

Connell, W. H. . Philadelphia, Pa.
Connelly, J. A. A. . New York City
Connor, E. H. Leavenworth, Kans.
Cope, E. L. . . San Francisco, Cal.
Corlett, W. G. . . . . . Oakland, Cal.
Crocker, H. S. . . . . . Denver, Colo.
Crooks, C. H. . . . . . New York City
Crosby, W. W. . . . . Baltimore, Md.

Darrow, W. J. . . . . New York City
Dater, P. H. . . . . . . Portland, Ore.
Davis, C. M. . . . . Fort Worth, Tex.
Davis, Frederick. . Davenport, Cal.
Davis, F. H. . . San Francisco, Cal.
Davis, G. R. . . . . Sacramento, Cal.
Dennis, H. W. . . Los Angeles, Cal.
Derleth, Charles, Jr. Berkeley, Cal.
Dewell, H. D. . San Francisco, Cal.
Deyo, S. L. F. . . . . New York City
Dibert, H. M. . . . . Troy, N. Y.
Dillman, G. L. San Francisco, Cal.
Doble, W. A. . San Francisco, Cal.
Dockweiler, J. H., San Francisco, Cal.
Dodge, J. L. . . . . . . Woodland, Cal.
Duncan, Lindsay. . . . McGill, Nev.
Durham, H. W. . . . New York City
Duryea, Edwin, Jr., San Francisco, Cal.
Dygert, H. I. . . . . . . Berkeley, Cal.

Eaton, Frederick, Los Angeles, Cal.
Eavenson, H. N. . . . Gary, W. Va.
Eberly, V. A. . . . . Chicago, Ill.
Eckart, N. A. San Francisco, Cal.
Eddy, A. J. . . . . Berkeley, Cal.
Egbert, Warren, San Francisco, Cal.
Elliott, G. A. M., San Francisco, Cal.
Enger, A. L. . . . . Tucson, Ariz.
Etcheverry, B. A. . . Berkeley, Cal.

Fain, J. R. . . . . Jellico, Tenn.
Filley, O. D., Manila, Philippine Islands
Fish, J. C. L., Stanford University, Cal.
Flaa, I. E. . . . . San Francisco, Cal.
Fletcher, A. B. . . Sacramento, Cal.
Foote, A. D. . . . Grass Valley, Cal.
Foote, F. S., Jr. . . . . Berkeley, Cal.
Freeman, John R. Providence, R. I.
Frickstad, W. N. . . . Oakland, Cal.
Frisell, E. H. . San Francisco, Cal.
Fulcher, R. E. San Francisco, Cal.
Fuller, G. W. . . . . . New York City

Galloway, J. D., San Francisco, Cal.
Garrison, F. L. . Philadelphia, Pa.
Gaston, L. P. . . . . . New York City
Gaston y Rosell, F. J., Havana, Cuba
Gay, F. W. . . . San Francisco, Cal.
Gearhart, W. S. Manhattan, Kans.
Gerry, M. H., Jr. . . Helena, Mont.
Gester, W. B. . San Francisco, Cal.
Giddings, Frederick, Lake Charles, La.
Gideon, Abraham, Manila, Philippine Islands
Gifford, R. L. . . . . Chicago, Ill.
Gilman, C. E. . San Francisco, Cal.
Gray, H. F. . . . . Palo Alto, Cal.
Griewank, A. C., San Francisco, Cal.
Griswold, H. S. . . . . Berkeley, Cal.
Griswold, L. S. San Francisco, Cal.
Grunsky, C. E. San Francisco, Cal.

Haas, E. F. . . San Francisco, Cal.
Haehl, H. L. . San Francisco, Cal.
Hall, C. R. . . . San Francisco, Cal.
Hall, H. H. . . San Francisco, Cal.
Hammatt, W. C., San Francisco, Cal.
Hanique, J. E. . . . . . Oakland, Cal.
Harding, S. T. . . . . . Berkeley, Cal.

Hardison, A. C. Santa Paula, Cal.
Harris, F. S. M.....Oakland, Cal.
Harroun, P. E. San Francisco, Cal.
Haskell, E. E.......Ithaca, N. Y.
Hawley, J. B....Fort Worth, Tex.
Hawley, R. S....Emeryville, Cal.
Hawley, R. W......Berkeley, Cal.
Hays, J. C....San Francisco, Cal.
Hazen, Allen......New York City
Hedges, S. H......Seattle, Wash.
Herrmann, F. C., San Francisco, Cal.
Herron, G. M.....Palo Alto, Cal.
Hess, J. S....San Francisco, Cal.
Hindes, S. G..San Francisco, Cal.
Hohl, L. L........Sausalito, Cal.
Holly, J. B...San Francisco, Cal.
Holmes, H. C. San Francisco, Cal.
Homberger, Heinrich, Mill Valley, Cal.
Hood, William, San Francisco, Cal.
Horton, J. W....Sacramento, Cal.
Howe, J. M........Houston, Tex.
Huber, W. L..San Francisco, Cal.
Hunt, Chas. Warren, New York City
Hunt, L. E.....Kansas City, Mo.
Hunter, T. B..San Francisco, Cal.
Hurlbut, W. W..Los Angeles, Cal.
Hyde, C. G........Berkeley, Cal.

Jacobs, J. L.......Houston, Tex.
Jamieson, J. Q....Portland, Ore.
Johnson, F. M.....Seattle, Wash.
Jorgensen, L. R., San Francisco, Cal.
Jubb, S. A.......San Pedro, Cal.
Judell, Adolph. San Francisco, Cal.

Kelton, F. C.......Tucson, Ariz.
Kempkey, Augustus, San Francisco, Cal.
Kibbe, A. S........Berkeley, Cal.
Kitts, J. A...Meadow Valley, Cal.
Knox, S. L. G. San Francisco, Cal.
Kower, Hermann...Berkeley, Cal.
Kriegsmann, E. F., San Francisco, Cal.
Kromer, C. H....Sacramento, Cal.

Lane, J. H....San Francisco, Cal.
Lebedeff, M. N.....Denver, Colo.
Legaré, B. P..San Francisco, Cal.
Loweth, C. F........Chicago, Ill.
Lundgren, Leonard. Portland, Ore.
Lydon, W. A........Chicago, Ill.
Lyman, R. R., Salt Lake City, Utah

MacGregor, R. A..New York City
McClintock, J. Y. Rochester, N. Y.
McGlashan, H. D...Berkeley, Cal.
McGonigle, C. J....Portland, Ore.
McLain, L. R..St. Augustine, Fla.
McMeekin, C. W., San Francisco, Cal.
McMillan, J. G....San José, Cal.
McMillan, W. B....San José, Cal.
McWethy, LeRoy, San Francisco, Cal.
Maddock, Thomas..Phœnix, Ariz.
Madison, J. T. San Francisco, Cal.
Manson, Marsden....Bellota, Cal.
Markwart, A. H., San Francisco, Cal.
Marshall, R. B. Washington, D. C.
Martin, C. D........Merced, Cal.
Martin, J. W....Long Beach, Cal.
Marx, C. D., Stanford University, Cal.
Maughmer, Carl. Sacramento, Cal.
Means, T. H..San Francisco, Cal.
Mehren, E. J......New York City
Meredith, Wynn, San Francisco, Cal.
Merriman, Mansfield, New York City
Metcalf, Leonard....Boston, Mass.
Monroe, R. A. San Francisco, Cal.

Moran, R. B. . San Francisco, Cal.
Morgan, J. H. San Francisco, Cal.
Morris, C. C. . San Francisco, Cal.
Mower, H. C. . . . . Tuscaloosa, Ala.
Muchemore, H. L.,
San Francisco, Cal.
Muckleston, H. B.,
Calgary, Alberta, Canada
Muhs, F. R. . . San Francisco, Cal.
Mulholland, William,
Los Angeles, Cal.
Mumm, Hans, Jr. . Everett, Wash.
Murphy, F. E.,
West New Brighton, N. Y.

Neville, C. W. J. New Orleans, La.
Newman, Emil. . North Fork, Cal.
Newman, Jerome,
San Francisco, Cal.
Nishkian, L. H.,
San Francisco, Cal.
Noble, H. A. . . . . Berkeley, Cal.
Norboe, P. M. . . . Sacramento, Cal.
Norris, R. V. A. Wilkes-Barre, Pa.
Norton, G. H. . . . . . Buffalo, N. Y.

Oakley, F. T. . . . . . . Oakland, Cal.
O'Hara, F. J. . San Francisco, Cal.
Older, Clifford. . . . Springfield, Ill.
O'Shaughnessy, M. M.,
San Francisco, Cal.
Overocker, D. W. . . . . Troy, N. Y.
Owens, J. M. . San Francisco, Cal.

Pardee, J. T. . . . . . Cleveland, Ohio
Paret, M. P. . San Francisco, Cal.
Parker, C. J. . . . . . New York City
Parsons, A. T. San Francisco, Cal.
Parsons, M. G. . . . . Pasadena, Cal.
Peck, M. H. . . . . Berkeley, Cal.
Pegram, G. H. . . . New York City
Petit, C. W. . . . . Ventura, Cal.
Plant, F. B. . . San Francisco, Cal.
Pope, C. S. . . . . . Los Angeles, Cal.
Popert, W. H. San Francisco, Cal.
Posey, G. A. . . . . . . Piedmont, Cal.
Poss, V. H. . . San Francisco, Cal.
Pracy, G. W. . San Francisco, Cal.
Pulligny, J. L. de. . Paris, France

Randolph, J. R. . . Blacksburg, Va.
Reddick, J. B. . . . . . Berkeley, Cal.
Redfield, C. M. . Des Chutes, Ore.
Reed, R. J. . . . . Los Angeles, Cal.
Rhodes, C. I. . . . . . . Berkeley, Cal.
Rhodin, C. J. San Francisco, Cal.
Rice, P. D. . . . . San José, Cal.
Richardson, Clifford,
New York City
Ricketts, L. D. . . . . Warren, Ariz.
Riffle, Franklin,
San Francisco, Cal.
Robson, F. T. San Francisco, Cal.
Robson, R. E. . . Atascadero, Cal.
Ropes, Horace. Minneapolis, Minn.
Rushmore, D. B.,
Schenectady, N. Y.
Ryan, W. J. . . Snoqualmie, Wash.

St. John, R. U. San Francisco, Cal.
Sanders, W. H. . Los Angeles, Cal.
Saph, A. V. . . . . Berkeley, Cal.
Saunders, H. J.,
San Francisco, Cal.
Saunders, W. L. . . New York City
Schneider, E. J.,
San Francisco, Cal.
Sharon, J. J. H.,
San Francisco, Cal.
Shields, J. R. . . . . . Berkeley, Cal.
Shutts, F. O. . San Francisco, Cal.
Smith, G. E. P. . . . . Tucson, Ariz.
Snyder, C. H. . San Francisco, Cal.
Soulé, E. L. . . San Francisco, Cal.
Southworth, E. A.,
Mill Valley, Cal.
Spencer, Herbert. . New York City
Stava, William,
San Francisco, Cal.
Stearns, R. H. . . . . . Boston, Mass.

Stocker, L. W., San Francisco, Cal.
Strout, G. S. . . San Leandro, Cal.
Swain, G. F. . . . Cambridge, Mass.

Tappan, Roger. . . . Topsfield, Mass.
Taylor, J. T. . . Honolulu, Hawaii
Thom, Neil, Jr., San Francisco, Cal.
Thompson, Benjamin. Tampa, Fla.
Thompson, R. A., San Francisco, Cal.
Thurston, E. T., Jr., San Francisco, Cal.
Tibbetts, F. H. . . . . Berkeley, Cal.
Tillinghast, F. H... Lahonton, Nev.
Tinsley, R. B. . Big Stone Gap, Va.
Trout, H. E. . . . . Sayre, Pa.
Trowbridge, A. L., San Francisco, Cal.

Uhlig, Carl. . . San Francisco, Cal.

Van Norden, R. W., San Francisco, Cal.
van Rensselaer, Allen, San Rafael, Cal.
Van Suetendael, A. O., Albany, N. Y.
Vensano, H. C., San Francisco, Cal.
Villalon, J. R. . . . . Havana, Cuba
Voorhees, I. S., Klamath Falls, Ore.

Wadsworth, H. H., San Francisco, Cal.
Wagoner, Luther, San Francisco, Cal.
Walsh, J. J. . San Francisco, Cal.
Waltman, W. D. . . . Casper, Wyo.
Warrington, H. E., Los Angeles, Cal.
Webster, G. S. . Philadelphia, Pa.
Wells, J. B. . . . . . Palo Alto, Cal.
White, F. G. . San Francisco, Cal.
Whitney, H. A. . . . Tacoma, Wash.
Wilder, A. D. . . . . . Berkeley, Cal.
Williams, Cyril, Jr., San Francisco, Cal.
Williams, G. S. . Ann Arbor, Mich.
Wilson, H. M. . . . Pittsburgh, Pa.
Wing, C. B. . . . . . Palo Alto, Cal.
Wiskocil, C. T. . . . Berkeley, Cal.
Wolf, J. H. G., San Francisco, Cal.
Wrightson, F. G., Jr., Sacramento, Cal.

SOCIETY ITEMS OF INTEREST

Report of Committee on Re-districting the Society*

NOVEMBER 5TH, 1915.

TO THE BOARD OF DIRECTION:

Your Committee appointed to take up the matter of the Re-districting of the Society begs leave to report as follows:

A great many studies have been made of various groupings of States to provide for the subdivision of the territory covered by the membership of the Society into 12 Districts, outside of the Resident District, to comply with the amendment to the Constitution recently adopted.

An effort was made to confine the boundaries of each District in all cases to the boundary lines of States, but in the arrangement recommended this has been departed from in two instances.

The question of making one District of the whole of Canada has been carefully considered, but was abandoned in favor of the recommended plan for two reasons, first on account of the great extent of territory involved, and second because in the proposed arrangement Mexico and the Provinces of Canada are brought into the Districts in the same way as any of the United States, thus recognizing the fact that those who live in Canada and Mexico are American, and therefore should be dealt with as are all other members of the American Society of Civil Engineers.

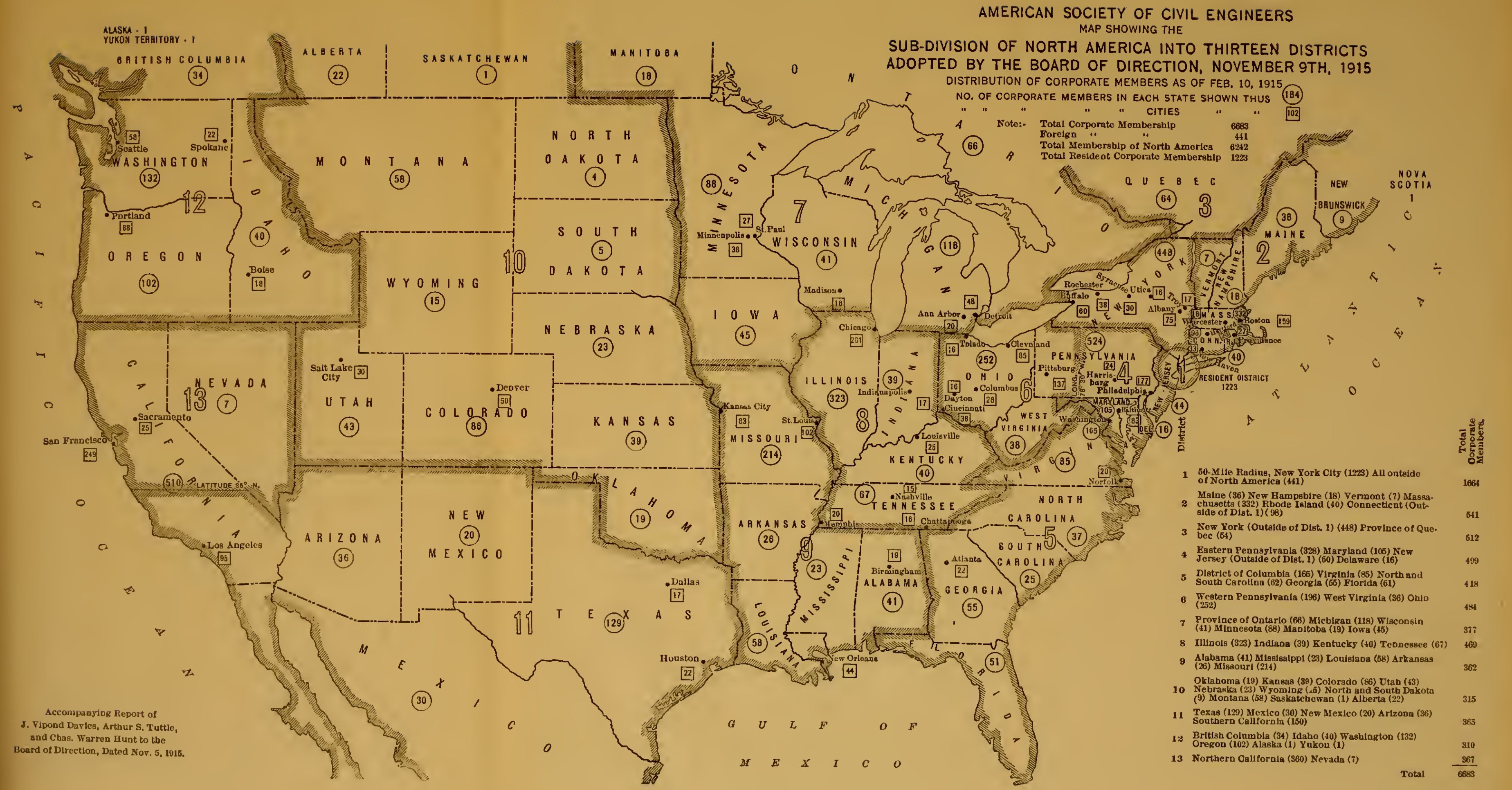
For the purpose of this study the total number of Corporate Members in the Society and in the various localities has been taken as of February 10th, 1915, when the last List of Members was issued. At that time:

The total Corporate Membership was.........	6 683		
The total foreign Corporate Membership.....	441		
The total Corporate Membership in North America...		6 242	
The total Resident Corporate Membership..........		1 223	
Total in North America outside of the Resident District....			5 019

In the plan which your Committee offers for the consideration of the Board, it is suggested that eventually, when the Constitution is next amended, that all members of the Society not resident in North America be attached to District No. 1, without, however, paying any additional dues.

It is believed that such foreign membership can be represented better by being attached to the Headquarters District than if separated in the manner which has been in force heretofore. For instance, at the present time the New England District (No. 2) includes Europe

* On November 9th, 1915, the Board of Direction received this report and adopted all the recommendations of the Committee. The districts, therefore, are fixed as stated in this report and shown on the map.


AMERICAN SOCIETY OF CIVIL ENGINEERS
MAP SHOWING THE
SUB-DIVISION OF NORTH AMERICA INTO THIRTEEN DISTRICTS
ADOPTED BY THE BOARD OF DIRECTION, NOVEMBER 9TH, 1915
DISTRIBUTION OF CORPORATE MEMBERS AS OF FEB. 10, 1915
NO. OF CORPORATE MEMBERS IN EACH STATE SHOWN THUS (184)
" " " " " CITIES " " [102]
Note:- Total Corporate Membership 6683
Foreign " " 441
Total Membership of North America 6242
Total Resident Corporate Membership 1223
ALASKA - 1
YUKON TERRITORY - 1
District
Total Corporate Members.
1 50-Mile Radius, New York City (1223) All outside of North America (441) 1664
2 Maine (36) New Hampshire (18) Vermont (7) Massachusetts (332) Rhode Island (40) Connecticut (Outside of Dist. 1) (98) 541
3 New York (Outside of Dist. 1) (448) Province of Quebec (64) 512
4 Eastern Pennsylvania (328) Maryland (105) New Jersey (Outside of Dist. 1) (60) Delaware (16) 499
5 District of Columbia (165) Virginia (85) North and South Carolina (62) Georgia (55) Florida (61) 418
6 Western Pennsylvania (196) West Virginia (36) Ohio (252) 484
7 Province of Ontario (66) Michigan (118) Wisconsin (41) Minnesota (88) Manitoba (19) Iowa (45) 377
8 Illinois (323) Indiana (39) Kentucky (40) Tennessee (67) 469
9 Alabama (41) Mississippi (23) Louisiana (58) Arkansas (26) Missouri (214) 362
10 Oklahoma (19) Kansas (39) Colorado (86) Utah (43) Nebraska (23) Wyoming (15) North and South Dakota (9) Montana (58) Saskatchewan (1) Alberta (22) 315
11 Texas (129) Mexico (30) New Mexico (20) Arizona (36) Southern California (150) 365
12 British Columbia (34) Idaho (40) Washington (132) Oregon (102) Alaska (1) Yukon (1) 310
13 Northern California (360) Nevada (7) 367
Total 6683
RESIDENT DISTRICT 1223
LATITUDE 36° N.
PACIFIC OCEAN
ATLANTIC OCEAN
GULF OF MEXICO
Accompanying Report of J. Vipond Davies, Arthur S. Tuttle, and Chas. Warren Hunt to the Board of Direction, Dated Nov. 5, 1915.

and Africa. The Southern District (No. 6) includes Mexico, Central America, South America, and the West India Islands. The Western District (No. 7) includes Hawaii, Asia, and Australasia.

If the re-districting recommended by the Committee is adopted by the Board, it will be easy to attach the various countries outside of North America temporarily to one or more of the Districts as laid out until such time as the Constitution is amended.

From the figures of membership given above, it will be seen that the 12 Districts should be arranged as nearly as possible to contain 418 members.

Your Committee believes that when these Districts are adopted they should remain for a number of years without change, and as a uniform growth is not probable, it is impossible to predict whether one District will grow faster than another, and, therefore, it is not believed that slight discrepancies in the number of members in each District as adopted for the first arrangement of Districts make any material difference.

In the arrangement of Districts which follows it will be noted that in two cases the lines of States have not been adhered to, the State of Pennsylvania being divided by a line running north and south, and the State of California being divided by a line running east and west. In the case of the first mentioned division, your Committee finds that the present District No. 4 includes Pennsylvania, Delaware, Maryland, Ohio, and the District of Columbia; that two large centers are at its easterly and westerly ends (Philadelphia and Pittsburgh), and it seems desirable to add that part of New Jersey not included in the Resident District to the District which includes Philadelphia, because members residing at Atlantic City, Camden, and, in fact, in any part of Southern New Jersey, are really tributary to Philadelphia. Similarly, in California, the residents of Los Angeles and San Diego are a long way from those residing in San Francisco, Sacramento, and the northern part of the State, and much more nearly in touch with members in Arizona, New Mexico, etc.

A sketch map is attached hereto showing the arrangements of Districts recommended by your Committee. They are as follows:

	Total Corporate Members.	Total.
District No. 1.—Resident District	1 223	
Add all members not residing in North America	441	1 664
District No. 2.—New England States................	531	
New Brunswick and Nova Scotia....	10	541
Carried forward................		2 205

		Total Corporate Members.	Total.
	Brought forward		2 205
District No. 3.—	New York (except as included in District No. 1)	448	
	Province of Quebec, Canada	64	512
District No. 4.—	Eastern Pennsylvania (East of W. Longitude 78° 30′)	328	
	New Jersey (except as included in District No. 1)	50	
	Maryland	105	
	Delaware	16	499
District No. 5.—	District of Columbia	165	
	Virginia	85	
	North Carolina	37	
	South Carolina	25	
	Georgia	55	
	Florida	51	418
District No. 6.—	Western Pennsylvania (West of W. Longitude 78° 30′)	196	
	West Virginia	36	
	Ohio	252	484
District No. 7.—	Michigan	118	
	Wisconsin	41	
	Iowa	45	
	Minnesota	88	
	Manitoba, Canada	19	
	Ontario, Canada	66	377
District No. 8.—	Illinois	323	
	Indiana	39	
	Kentucky	40	
	Tennessee	67	469
District No. 9.—	Alabama	41	
	Mississippi	23	
	Louisiana	58	
	Arkansas	26	
	Missouri	214	362
	Carried forward		5 326

		Total Corporate Members.	Total.
	Brought forward		5 326
District No. 10.	Oklahoma	19	
	Kansas	39	
	Colorado	86	
	Utah	43	
	Nebraska	23	
	Wyoming	15	
	North Dakota and South Dakota	9	
	Montana	58	
	Saskatchewan	1	
	Alberta	22	315
District No. 11.	Texas	129	
	Mexico	30	
	New Mexico	20	
	Arizona	36	
	Southern California (South of N. Latitude 36°)	150	365
District No. 12.	Idaho	40	
	Washington	132	
	Oregon	102	
	Alaska	1	
	British Columbia	34	
	Yukon	1	310
District No. 13.	Northern California (North of N. Latitude 36°)	360	
	Nevada	7	367
	Total		6 683

It will be noticed that Districts Nos. 2, 3, 4, 6, and 8, contain somewhat in excess of the number required for an equal distribution, No. 5 being exactly what is called for by such distribution. Nos. 7, 9, 10, 11, 12, and 13, all of which Districts are in the West and somewhat sparsely settled, contain somewhat less than the average number, but it is obvious, as before stated, that an absolutely equal subdivision is not possible, and your Committee feels, as a result of its study of the question, that it would not be wise, in the best interests of the Society, to consider too closely the number of members in each District, but that greater weight should be attached to the professional and sectional interests of the communities which will be formed by each subdivision.

As far as it has appeared possible the grouping of the 12 Districts has been made with regard to the existing 15 Local Associations of Members, and the endeavor has been to form the Districts so that large cities having a considerable membership may balance fairly with the scattered membership in the less populated Districts. It will be noticed also that the Districts having the largest membership are those in which there are large cities and where conditions are more settled, thereby giving to the growing, less populated, and larger areas, the advantage in respect to numbers.

While, as before stated, the Committee believes that when once established these Districts must probably remain for a number of years as fixed, it recommends that in the proposed revision of the Constitution a provision be made that whenever necessary it may be possible for the Board of Direction to modify and re-arrange the Districts should the conditions call for such re-arrangement.

All of which is respectfully submitted,

J. Vipond Davies, *Chairman*,
Arthur S. Tuttle,
Chas. Warren Hunt.

Memorandum to Accompany Report on Re-districting.

There are the Following Local Associations:

Name.	Membership.	Territory included in Association.	New District in which located.
Atlanta Association............	*	Indefinite †......................	No. 5.
Baltimore Association..........	80	Maryland..........................	No. 4.
Cleveland Association..........	45	Eastern Ohio......................	No. 6.
Colorado Association...........	75	Colorado ‡........................	No. 10.
Louisiana Association..........	35	Louisiana.........................	No. 9.
Northwestern Association......	§	North Dakota, South Dakota, Minnesota, part of Wisconsin, and Upper Peninsula of Michigan............................	No. 7 partly. No. 10 partly.
Philadelphia Association.......	112	Delaware and Pennsylvania, east of Susquehanna River...	No. 4.
Portland, Ore., Association.....	60	Radius of 100 miles from Portland. Ore......................	No. 12.
San Francisco Association......	188	California, except as covered by Southern California Association.........................	No. 13.
St. Louis Association...........	65	Radius of 200 miles from St. Louis, Mo.....................	No. 9.
Seattle Association.............	80	Indefinite........................	No. 12.
Southern California Association.	93	Limited on north by "topographical division"..........	No. 11.
Spokane Association............	30	Radius of 250 miles from Spokane, Wash.....................	No. 12.
Texas Association...............	68	Texas.............................	No. 11.

* Was stated that there are 24 members in Atlanta.

† President Dallis stated "Any member of the American Society, regardless of where he lives, may become a member of the Atlanta Association."

‡ Not limited strictly to Colorado.

§ President Cappelen stated that he did not know the exact membership. Forty members present at each of two meetings.

Report of Representatives of the American Society of Civil Engineers Appointed in Connection with the Constitutional Convention of the State of New York

November 8, 1915.

To the Board of Direction of the
American Society of Civil Engineers.

Gentlemen: Pursuant to a resolution adopted by the Board at its January meeting, the undersigned were appointed as a Committee to represent the American Society of Civil Engineers at the Constitutional Convention of the State of New York.

Your Committee begs to report that after formulating certain tentative suggestions concerning amendments in the organic law, the adoption of which it was believed would result in substantially improving the management of those functions of the State Government with which the engineering profession is most directly concerned, and after making these suggestions a subject of debate at a special meeting of the Society, an attempt was made to enlist the support of other National Engineering Societies in the movement, as a result of which a Joint Committee was formed embracing in its membership your Committee together with representatives of the American Institute of Electrical Engineers, the American Society of Mechanical Engineers, the American Institute of Consulting Engineers, the New York Section of the American Institute of Mining Engineers, the Municipal Engineers of the City of New York, and the Brooklyn Engineers' Club.

The work thereafter accomplished is fully outlined in a report from the officers of this Joint Committee, which was approved by it at a meeting held on November 5th, a copy of which is appended hereto as a part of this report.

Your Board has made an appropriation of $500 to meet the expenses of this Committee, and the books of the Secretary show that the total expenditures to date which have been charged against this account amount to $79.57. The total amount expended by the Committee, including the disbursements made by this Society, amount to $369.40, with a few small accounts outstanding. Under the apportionment of the expense as agreed upon by the Joint Committee, our share will total about $100, the exact amount to be determined by the Treasurer of the Committee within the next thirty days, when it is understood that all of the bills will have been rendered and subscriptions paid.

The Committee would recommend that the Secretary be authorized to pay such balance as may yet be due upon the presentation of a bill from the Treasurer of the Committee, and that the Committee be now discharged.

At the last meeting of the Joint Committee the Chairman was directed to advise the several Societies which participated in it that information had been presented which led the Committee to believe that several of the principles advocated had met with a sufficiently favorable reception to warrant an expectation that they would obtain additional advocates in future campaigns of this character, and that the members of this Committee are of the opinion that "its labors should not be lost, and that a similar Committee should be appointed to continue the work".

Your Committee concurs in the suggestion and recommends to the Board that it appoint a Committee to join with a similar Committee from other Societies to represent the profession in matters relating to Constitutional Convention or Legislative amendments in the laws of New York.

Respectfully,

ARTHUR S. TUTTLE,
HENRY W. HODGE,
ALFRED D. FLINN.

FULL REPORT OF JOINT COMMITTEE

NOVEMBER 5, 1915.

TO THE COMMITTEE OF ENGINEERS,
REPRESENTING NATIONAL AND LOCAL
PROFESSIONAL ENGINEERING SOCIETIES
AT THE CONSTITUTIONAL CONVENTION
OF THE STATE OF NEW YORK.

GENTLEMEN:

In accordance with the directions given at the meeting of the Committee held on August 30, 1915, the undersigned present the following report concerning the work done by the Committee in the matter of placing suggestions from the engineering profession relative to the organization of such functions of government as relate to Public Works and Public Utilities before the members of the Constitutional Convention of the State of New York.

This movement originated in a communication from E. J. Mehren, of the Am. Soc. C. E., to the Board of Direction of that Society calling attention to the desirability of making an effort to secure, through the medium of this Convention, "proper recognition of the Engineer's place in public administration," as a result of which the Board authorized the appointment on January 26th of a Committee of three of its members to represent the Society at the Convention. This Committee after several weeks' consideration agreed upon certain basic principles which it believed should be observed in formulating provisions concerning State Government in order to

insure intelligent and businesslike direction, and it also proposed in detail the modifications which, in its judgment, were required in the existing Constitution in order to bring about the practical application of these principles. Realizing that this important subject, which so directly concerned the entire profession, was one requiring most mature consideration from every viewpoint, the Committee felt that a strong effort should be made to secure the views of their fellow members and also to obtain the general support needed in order to insure the success of the movement. To this end, the American Society of Civil Engineers devoted the evening of March 17th to a discussion of the proposition formulated by the Committee, which for this purpose was so drafted as to bring out the views of those present concerning several other points which the Committee had considered but on which no definite conclusion had been reached. To this meeting the resident members of all the National Engineering Societies were informally invited and freely extended the privileges of the floor. As a result of this discussion, which brought out many valuable suggestions, and of advice communicated from members at a distance whose views were courted, the Committee promptly prepared its final recommendations for submission, but before taking the latter step it decided first to seek the informal criticism of members of the principal other National Engineering Societies to the end that the proposition might be so shaped as to insure that, in so far as practicable, it would meet the viewpoint of each, none of them at that time having made a definite move to participate actively in this work. This step resulted in an informal conference on April 1st with representative members of these Societies, all of whom appreciated the desirability of co-operation as well as the necessity of immediate action, April 15th having been fixed as the date for convening the Convention. The conference was followed on April 6th by the formal organization of this Committee representing several National and Local Engineering Societies, the election of officers, and the adoption of a definite policy both as to the principles to be observed and as to the specific recommendations to be made to the Convention as well as the form in which they were to be presented.

As the Committee was finally organized, the Societies represented and their delegates were as follows:

American Society of Civil Engineers:	Arthur S. Tuttle, Henry W. Hodge, Alfred D. Flinn.
American Institute of Electrical Engineers:	Gano Dunn, Calvert Townley, William McClellan.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS:	Arthur M. Greene, Jr., E. Gybbon Spilsbury, Charles Whiting Baker.
AMERICAN INSTITUTE OF CONSULTING ENGINEERS:	Ralph D. Mershon, Alten S. Miller, Charles W. Leavitt, Jr.
NEW YORK SECTION, AMERICAN INSTITUTE OF MINING ENGINEERS:	W. L. Saunders, Benjamin B. Lawrence, J. Parke Channing.
MUNICIPAL ENGINEERS OF THE CITY OF NEW YORK:	Alfred D. Flinn, George W. Tillson, Ernest P. Goodrich.
BROOKLYN ENGINEERS' CLUB:	Nelson P. Lewis, William W. Brush, Jacob S. Langthorn.

The Committee was also further strengthened by the election of Calvin W. Rice as Secretary-Treasurer, and E. J. Mehren as Assistant Secretary.

In order that as many of the Societies as practicable might be formally included in the official representation, it was found necessary to delay the presentation of our views to the Convention until April 20th, on which date a copy of the recommendations was sent to each member of the Convention and at the same time steps were taken to place them in a direct way before Chairman Elihu Root. The latter was accomplished on May 7th when a Committee of five of your members were accorded the privilege of a lengthy interview and a full discussion of the proposition without eliciting adverse criticism other than as to the recommendations for multi-headed commissions designed to secure continuity of policy; the undesirability of the latter provision was urged by Chairman Root on the ground that administration of this character failed to definitize responsibility and to insure its assumption by the entire membership.

Following the division of work adopted by the Convention, our Committee provided for the creation of six sub-committees to take charge of the matter of pressing our views concerning the various questions involved, as follows:

CANALS:	E. GYBBON SPILSBURY, *Chairman,*
CONSERVATION:	CHARLES W. LEAVITT, JR., *Chairman,*
JUDICIARY:	ALFRED D. FLINN, *Chairman,*
INDUSTRIAL INTERESTS:	HENRY W. HODGE, *Chairman,*
GOVERNOR AND OTHER STATE OFFICERS:	NELSON P. LEWIS, *Chairman,*
PUBLIC UTILITIES:	CHARLES WHITING BAKER, *Chairman.*

At the same time Professor Arthur M. Greene, Jr., was asked to act as the special representative at Albany, N. Y., E. J. Mehren was charged with the duty of keeping the general and all the sub-committees apprised of developments in the Convention, and the Chairman and Secretary were constituted a Publicity Committee. Each sub-committee was also given power to add to its membership as found necessary in order to bring about a successful outcome of its work, and through this means valuable assistance was rendered to these committees by Messrs. Mortimer G. Barnes, William H. Burr, E. A. Fisher, A. B. Fry, Charles Hansel, Carl A. Meissner, George A. Ricker, and James E. Sague.

The records show that eight formal meetings of our Committee were held during the Convention, which adjourned on September 10th, while the work of some of the officers and sub-committees involved almost constant co-operative effort and conference. At an early date it became evident that to insure the consideration of such of our propositions as were of a character demanding positive action, it would be necessary to formulate them in such terms as to permit of direct incorporation in the Constitution as revised, while those more in the nature of suggestions conditional upon possible action by the Convention along the lines covered, were made the subject of correspondence with the Chairmen of the Convention Committees on Canals, Judiciary, and Industrial Interests, in which our views were pressed to attention. To meet the former end, the proposed amendments were drafted by the sub-committees on Governor and other State Officers, Conservation, and Public Utilities, but the first named committee was the only one which succeeded in perfecting its draft at a date early enough to secure a hearing before the Convention Committee. At this hearing the views relative to the organization of the proposed Department of Engineering and Public Works were fully presented and a strong argument was offered in their support. This sub-committee, together with those on Public Utilities, and Conservation, also drafted letters to the respective Convention committees briefly and vigorously setting forth the views of the general committee, while a similar letter on all of these subjects was sent to Chairman Root by the officers of your committee, in which the measures proposed by the Convention were reviewed and compared with those which we urged. Copies of these letters were circularized in the Convention and sent to the press, and the views therein expressed were backed up by individual work on the part of members of our committee to influence their acquaintances in the membership of the Convention. These efforts were also supported by the Professional Engineers of Rochester, who on May 19th agreed upon recommendations almost identical with our own, and by the entire engineering press, which made a strong effort to secure general interest

on the part of the profession in furthering the work which the Committee had undertaken.

A review of these efforts fails to disclose any weakness in committee organization, method, or interest in bringing expert views before a body of men presumably seeking the best advice that was obtainable, but the recommendations adopted by the Convention upon its adjournment fail to meet our views to the extent desired and which we believe we had reason to expect.

As a basic principle, the Committee had assumed that the term of office to be fixed for the Governor would be lengthened from two to four years, and the recommendations made concerning the term of office for the three Commissioners proposed for the Department of Engineering and Public Works and for the five Commissioners proposed for a single Department of Public Utilities, were intended to harmonize with a long term for the Chief Executive. The Convention, however, determined to retain the term of two years for the Governor, thus weakening to a large extent the plea for long terms of office for commissioners without seriously prejudicing the "concentrated responsibility" which it was also urged should be placed on elected officers. As a partial offset and in evident recognition of the need of continuing in office heads of departments charged with important duties which could only be performed after acquiring complete familiarity with their nature and scope, the Convention omitted any provision as to the term for which such officers should be appointed, leaving the matter of both appointment and removal entirely within the province of the Governor. It is evidently expected that the requirement as to some definite act in order to remove a department head will have the effect of continuing in office men who for political reasons probably would not be re-appointed.

Taking up the more important recommendations of the Committee in their order, the results accomplished may be briefly stated as follows:

Engineering Service.—Under the provisions heretofore made for the State Government the engineering functions were scattered among a large number of departments, and the State Engineer was elected by popular vote. The centralization of all of these functions into the Department of Engineering and Public Works was urged by the Committee. This effort had a successful outcome, and the Department of Public Works proposed by the Convention and headed by a Superintendent of Public Works is to be given complete control over every present and future engineering activity.

Public Utilities.—It was urged by the Committee that this Department, which had not been previously given Constitutional recog-

nition, should be created and should be headed by five commissioners, two of whom should be professional engineers. The Convention decided to retain the two separate departments now existing and to perpetuate the existing Public Service Commissions without restriction of any sort as to the character of their membership or as to their duties, these being left subject to legislative action.

Conservation.—It had been hoped by the Committee that the prohibitions in the present Constitution against carrying on scientific forestry in its proper economic sense and the reasonable development of the natural resources of the State might be removed. No progress, however, was made in this direction, it evidently being believed by the Convention that any breaking down of existing barriers along these lines might result prejudicially to the State, and that it was preferable to incur the wastefulness of the present methods rather than to risk the possible misuse of such power as might be vested in a commission if the present prohibitions were removed. The Convention proposes to charge the responsibility for this work to a nine-headed Conservation Commission, each member of which will have a nine-year term of office without compensation. There can be little doubt but that this latter provision will tend to insure the inauguration of a policy from which the most that can be hoped will be extreme conservatism.

Court of Claims.—It is understood that the work of this court is devoted very largely to investigations of an engineering character, and it was urged that, if it were given recognition in the Constitution, at least one-third of its membership should be professional engineers. This recommendation evidently found no weight in a Convention composed of 168 delegates, of whom only one was a practicing engineer, while by far the greater percentage of the remainder were lawyers. As drafted, the recommendations of the Convention provide for the creation of this court, but no specification is drawn as to the personnel of its membership, other than to provide for continuing the present incumbents in office.

Court Commissioners.—It was urged by our Committee that there should be no direct or implied prohibition against legislation which would permit of referring technical matters made the subject of litigation to a referee expert in such matters. This court, the creation of which is proposed by the Convention, is to be charged with the duty of taking testimony and determining upon awards in proceedings for condemning property required for public use. The value of the services of the professional engineer in work of this character is unquestioned, but nevertheless the Convention saw fit to lay down a prohibition against the appointment to this body of other than lawyers with at least ten years' active practice.

Industrial Board.—The Committee asked that in case an Industrial Board were created, its membership should include at least one professional engineer. In the proposed Constitution provision is made for such a Board, but no qualifications are named as to the character of appointees to it.

Incidentally, reference might be made to the requirement proposed by the Convention that the Department of Public Works must prepare and file an estimate of cost for an improvement, and execute a certificate as to its necessity, prior to its consideration by the Legislature, to the provision for submission by the Governor of a systematic budget relative to appropriations, and also to the prohibition against the issuance of bonds extending over a period greater than the life of the improvement for the construction of which they may be issued. In all of these particulars it is clear that a long step in advance of past practice has been formulated and one which could be adopted with great advantage.

It seems fair to conclude that the lack of more complete success in our undertaking is due to the absence of persistent and constant representation in the Convention by some one who could devote his whole time to pressing actively our views before the appropriate committees, to the lateness of the date on which our program was completed in detail, and to the fact that the profession was so scantily represented in the Convention. In this connection it would also seem proper to allude to the excellent work done by the sub-committees, the members of which devoted themselves to their assignments in an aggressive way and at all times rendered most valuable service in the evident belief that the great public benefit which it was hoped would result, justified them in sacrificing their time as well as their personal convenience to this work, as was very frequently the case.

We are attaching hereto copies of the recommendations originally presented, together with the various formal communications presented to or circulated in the Convention. There is also appended a financial statement to date, showing the receipts and disbursements of the Committee.

It is hoped that this record of our experience may be helpful to the profession in other States where Constitutional revision or Legislative action is now or may later be under consideration.

Respectfully submitted.

ARTHUR S. TUTTLE,
Chairman.

CALVIN W. RICE,
Secretary-Treasurer.

E. J. MEHREN,
Assistant Secretary.

Communication from Committee of Engineers Representing National and Local Professional Engineering Societies

New York, April 20, 1915.

To the New York State
Constitutional Convention.

There are few functions of government of greater importance than the administration of Public Works and the regulation of Public Utilities. The former embraces such great undertakings as the new Barge Canal and the State highways. The latter includes the supervision of the corporations engaged in operations intimately connected with the prosperity, health and comfort of all our citizens.

Under the present Constitution, the way is open for each two-year administration to change completely the management of enterprises which involve operations extending over long terms of years and which can be carried out efficiently and economically only through adhering to well considered plans determined at the outset. Under current practice, changes often are due not to proven incapacity of superseded officials, nor to established merit of their successors, but rather to political expediency. In private business, changes of this character often result in bankruptcy. The principle of continuity was recognized when the Public Service law was enacted, but it has not been written into the Constitution, and until this shall have been done, the method of applying it will be subject to the varying views of the successive Governors and Legislatures.

Professional engineers are conversant with existing defects in the administration of public works and the regulation of public utilities. In order that the Convention might have the benefit of engineers' views, a public discussion was held in March in which New York members of national engineering societies participated. This has been followed by conferences of committees representing the national societies, and finally by the formation of a Joint Committee including in its membership the national and two local societies. The national societies have about 30,000 members, of whom nearly 6,000 are residents of New York State, while the local societies' membership is about 1,000. The statement of principles and the recommendations transmitted herewith express the views of these committees and we believe that they are entitled to careful consideration by the Convention.

Since engineering works and organizations are unfamiliar matters to many persons, and have not heretofore been given substantial recognition in our State Constitution, the necessity for our recommendations may not at first be fully apprehended. For this reason, and

because of the very large expenditures involved, and on account of the numerous interests affected, we earnestly and disinterestedly urge a very careful study of these governmental functions. Small as well as large investors are vitally concerned. Moreover, state finance and credit are deeply influenced by the wise and economical, or the ill-advised and inefficient administration of public works and regulation of public utilities. We are in accord with a conservative tendency in the Convention, and believe that the Constitution should be as brief as may be and should deal mostly with principles of government, rather than with details of organization, but we are persuaded that the technical developments of the period which has intervened since the present Constitution was adopted has introduced into government large problems in connection with public works and utilities requiring legislation which should be based on constitutional provisions.

We trust that these views will have careful consideration and will commend themselves to the Convention. We request opportunities to present before the committees to which these matters may be referred reasons in support of our recommendations. Our services are freely at the disposal of your committees to the end that a structure of efficient and economical State Government may be built on a firm and enduring basis.

Very respectfully,

Committee of Engineers:

Arthur S. Tuttle Henry W. Hodge Alfred D. Flinn	*for American Society of Civil Engineers*
Gano Dunn Ralph D. Mershon William McClellan	*for American Institute of Electrical Engineers*
Arthur M. Greene, Jr. Charles Whiting Baker E. Gybbon Spilsbury	*for American Society of Mechanical Engineers*
Ralph D. Mershon Charles W. Leavitt, Jr. Alten S. Miller	*for American Institute of Consulting Engineers*
Alfred D. Flinn George W. Tillson Ernest P. Goodrich	*for Municipal Engineers of The City of New York*
Nelson P. Lewis William W. Brush Jacob S. Langthorn	*for Brooklyn Engineers' Club*

ARTHUR S. TUTTLE, *Chairman,*
c/o American Society of
Civil Engineers,
220 West 57th St., New York.

CALVIN W. RICE, *Secretary,*
c/o American Society of
Mechanical Engineers,
29 West 39th St., New York.

E. J. MEHREN, *Assistant Secretary,*
c/o American Society of
Civil Engineers,
220 West 57th St., New York.

P. S.—A committee of the American Institute of Mining Engineers, consisting of W. L. Saunders, Benjamin B. Lawrence, and J. Parke Channing, collaborated in the preparation of the "principles" and "recommendations." Individually, the members of this committee approved them. Pending consideration by the Governors of the Institute, the committee is unable to subscribe officially.

PRINCIPLES TO BE OBSERVED IN AMENDING THE CONSTITUTION OF NEW YORK STATE.

The Constitution should insure:

(*a*) A short ballot, making possible close scrutiny of the qualifications of all candidates and concentrated responsibility of elected officers.

(*b*) Tenure of office for all elected officials, for heads of departments, and for bureau heads, of sufficient duration to attract competent men and to permit them to become increasingly efficient in the discharge of their duties.

(*c*) A continuing policy in the organization and control of all departments, so that appropriations may be spent most economically and work carried on most efficiently.

(*d*) Selection of heads of departments by appointment instead of election by popular vote.

(*e*) Ample opportunity for the proper development of natural resources.

(*f*) Recognition of the value of technical advice through the inclusion of professional Engineers in the membership of Departments or Courts where such advice is essential to the proper conduct of the work or adjudication of the matters involved.

RECOMMENDATIONS.

1. The elective office of the State Engineer and Surveyor should be abolished, and the duties should be transferred to the Department of Engineering and Public Works hereinafter proposed.

2. A Department of Engineering and Public Works should be created to be headed by three commissioners appointed by the Governor, each to have a twelve-year term of office so arranged that a vacancy will be created every four years immediately after the inauguration of a new Governor. Commissioners should be eligible for reappointment. At least one commissioner should be a professional engineer in good standing in his profession, and each should have had experience that would fit him for the duties of the office. At least one and not more than two commissioners should be residents of New York City.

This Department should have charge of public lands and boundary surveys; of buildings, parks, highways, canals, and other public works, including design, construction, maintenance, and operation; and of the conservation and development of State resources.

This Department should be divided into suitable bureaus. Each bureau charged with responsibility for engineering work should be headed by a Chief Engineer selected by the commissioners with sole regard to his peculiar fitness for the duties of the bureau.

3. A Department of Public Utilities should be created to be headed by five commissioners appointed by the Governor, each to have a ten-year term of office so arranged that a vacancy will be created every two years. Commissioners should be eligible for reappointment. At least two Commissioners should be professional engineers in good standing in the profession. Each appointee should have had experience in connection with public utilities which would fit him for the duties of the office. At least two and not more than three commissioners should be residents of New York City.

This Department should regulate and supervise all common carriers, all water supply, irrigation, drainage, gas, power, lighting, heating, intelligence-transmitting and other public utility corporations operating within the limits of New York State, including similar activities on the part of any other State department or political subdivision of the State.

This department should be divided into such bureaus as may be essential. Each engineering bureau should be administered by a Chief Engineer selected with sole regard to his peculiar fitness for the office, and he should have power, subject to the approval of the commissioners, to select and appoint such division engineers as are essential to the proper conduct of his office.

4. In case provision be made in the Constitution for the creation of one or more departments or commissions charged with responsibility for regulating, supervising, and inspecting buildings and the equipment thereof, trades, mines, industries or labor, each such department or commission should include in its membership at least one professional engineer in good standing in his profession. Commissioners

should be appointed by the Governor. Each appointee should have had experience and should possess qualifications fitting him to perform the duties essential to the department or commission.

5. In case provision be made in the Constitution for the creation of a Court or Board of Claims charged with responsibility for investigating claims against or on behalf of the State, one-third of the membership of such Court or Board should be made up of professional engineers in good standing in the profession, each to have practiced professional engineering for at least ten years and for at least five years to have had responsible charge of important engineering work either as to design or execution.

6. Removal by the Governor of any commissioner should be made only after the filing of charges and after affording the accused an opportunity to be heard in the matter, provided, however, that at any time within the first six months after making an appointment the Governor may exercise the power of summary removal. A successor to a commissioner who has been removed should be appointed to fill the unexpired term. Vacancies caused by removal or otherwise should be filled under conditions similar to those governing original appointment.

7. There should be no direct or implied prohibition against legislation which would permit the reference by a court of technical matters made the subject of, or incidental to litigation, to a referee expert in such technical matters, for the purpose of securing a determination concerning the facts, or which will prevent the court from selecting independent expert witnesses or advisers in matters of this character.

8. In order to make possible the reasonable development of State resources, the present Constitutional prohibition against the use or sale of land and cutting of trees within the limits of the Forest preserve should be removed in so far as such use or sale of land or cutting of trees is essential to such development (Article VII, Section 7), and similarly to permit the development of other natural resources by private enterprise, provision should be made for the condemnation of private property necessary to the construction and operation of works for irrigation, drainage, sanitation, water supply, agriculture, mining, forestry, or power development, through the declaration of such project as *for public use,* subject, however, to the superior right of the State or of a subdivision thereof to condemn the same property or a portion thereof for State or municipal purposes (Article I, Section 7).

(NOTE: Several reports, from sub-committees of the Committee of Engineers, which formed a part of the general statement of the Committee's activities, are not reproduced here but are on file in the Library of the Society, where they may be examined by those who are interested.—*Secretary.*)

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

January 5th, 1916.—8.30 P. M.—A regular business meeting will be held, and a paper by William P. Creager, M. Am. Soc. C. E., entitled "The Economical Top Width of Non-Overflow Dams", will be presented for discussion.

This paper was printed in *Proceedings* for November, 1915.

February 2d, 1916.—8.30 P. M.—This will be a regular business meeting. Two papers will be presented for discussion, as follows: "The Failure and Righting of a Million-Bushel Grain Elevator", by Alexander Allaire, M. Am. Soc. C. E.; and "Cohesion in Earth: The Need for Comprehensive Experimentation to Determine the Coefficients of Cohesion", by William Cain, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings.*

ANNUAL MEETING

The Sixty-third Annual Meeting will be held at the Society House, on Wednesday and Thursday, January 19th and 20th, 1916.

COMMITTEE OF ARRANGEMENTS

FREDERICK C. NOBLE,

W. W. BRUSH, J. O. ECKERSLEY,

GEORGE L. LUCAS, CHARLES WARREN HUNT.

The business meeting will be called to order at 10 o'clock on Wednesday morning. The Annual Reports will be presented, officers for the ensuing year elected, members of the Nominating Committee appointed, Reports of Special Committees presented for discussion, and other business transacted.

An excursion will be arranged for the afternoon of Wednesday, and in the evening there will be a reception, with dancing, at the Hotel Astor. Thursday will probably be devoted to an all-day excursion, the details of which will be announced later; and on Thursday evening there will be a Smoker at the Society House.

SPECIAL MEETING

A meeting, for discussion of the Semi-Final Report of the Special Committee on Materials for Road Construction, will be held at 10 A. M., on Friday, January 21st, 1916 (the day following the close of the Annual Meeting of the Society).

PROGRESS REPORTS OF SPECIAL COMMITTEES

Progress Reports of the Special Committees on Materials for Road Construction, on Steel Columns and Struts, on A National Water Law, and on Floods and Flood Prevention, are printed in this number of *Proceedings* with the Papers and Discussions.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In making a search it sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is small. This method is particularly useful when there are drawings or figures in the text, which would be very expensive to reproduce by hand.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings.*

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the

Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings.*

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M., every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, L. R. Hinman, 1400 West Colfax Ave., Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, at 12.30 P. M., at Clarke's Restaurant, 1632 Champa Street.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meeting)

October 9th, 1915.—The meeting was called to order at the Denver Athletic Club; President Field in the chair; L. R. Hinman, Secretary; and present, also, 22 members and 4 guests.

The minutes of the Annual Meeting of June 12th, and of the Special Meeting of June 25th, 1915, were read and approved.

The report of the Committee appointed to arrange entertainment at Colorado Springs, Colo., for members of the National Engineering Societies *en route* to the International Engineering Congress, at San Francisco, Cal., submitted by William A. Bartlett, Chairman, was read by the Secretary, and, on motion, duly seconded, the report was accepted and ordered filed.

On motion, duly seconded, the Committee was discharged with a vote of thanks from the Association for its excellent work.

The question of the weekly luncheons was briefly discussed, and, on vote, 12.30 P. M. was decided as the time most convenient for those present.

A paper by Mr. S. B. Williamson, entitled "The Reorganization of the Reclamation Service and Its Purpose", was presented by the author.

On motion, duly seconded, a vote of thanks was tendered Mr. Williamson for his interesting and instructive paper.

Adjourned.

Atlanta Association

The Atlanta Association of Members of the American Society of Civil Engineers was organized on March 14th, 1912. The Association holds its meetings at the University Club, Atlanta, Ga.

At the meeting of the Association on January 9th, 1915, the following officers were elected for the ensuing year: President, Park A. Dallis; First Vice-President, B. M. Hall; Second Vice-President, P. H. Norcross; Secretary-Treasurer, T. B. Branch.

Baltimore Association

On May 6th, 1914, the Baltimore Association of Members of the American Society of Civil Engineers was organized, a Constitution adopted, and the following officers were elected: J. E. Greiner, President; Francis Lee Stuart, First Vice-President; L. H. Beach, Second Vice-President; Harry D. Williar, Jr., Secretary-Treasurer; and Messrs. H. D. Bush, B. T. Fendall, B. P. Harrison, Calvin W. Hendrick, Oscar F. Lackey, M. A. Long, and A. A. Thompson, Directors.

At its meeting of September 2d, 1914, the Board of Direction considered and approved the proposed Constitution of the Baltimore Association of Members of the American Society of Civil Engineers.

Cleveland Association

The proposed Constitution of the Cleveland Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on January 6th, 1915.

The following officers have been elected: President, Willard Beahan; Vice-President, Robert Hoffmann; Secretary-Treasurer, George H. Tinker.

Louisiana Association

At the meeting of the Louisiana Association of Members of the American Society of Civil Engineers (New Orleans, La.), on April

14th, 1915, the following officers were elected for the ensuing year: J. F. Coleman, President; W. B. Gregory and A. M. Shaw, Vice-Presidents; Ole K. Olsen, Treasurer; and E. H. Coleman, Secretary.

Northwestern Association

The proposed Constitution of the Northwestern Association of Members of the American Society of Civil Engineers (St. Paul and Minneapolis, Minn.) was considered and approved by the Board of Direction of the Society on November 4th, 1914. F. W. Cappelen is President and R. D. Thomas, Secretary.

Philadelphia Association

The meetings of the Philadelphia Association of Members of the American Society of Civil Engineers are held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

The officers of the Association are as follows: President, Edward B. Temple; Vice-Presidents, Edgar Marburg and John Sterling Deans; Directors, J. W. Ledoux, H. S. Smith, Henry H. Quimby, and George A. Zinn; Past-Presidents, George S. Webster and Richard L. Humphrey; Treasurer, S. M. Swaab; and Secretary, W. L. Stevenson.

Portland, Ore., Association

At the Annual Meeting of the Association on September 28th, 1915, the following officers were elected for the ensuing year: President, J. P. Newell; First Vice-President, John T. Whistler; Second Vice-President, E. B. Thomson; Treasurer, Russell Chase; and Secretary, J. A. Currey.

St. Louis Association

The proposed Constitution of the St. Louis Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on October 7th, 1914.

The following officers have been elected: President, J. A. Ockerson; First Vice-President, Edward E. Wall; Second Vice-President, F. J. Jonah; Secretary-Treasurer, Gurdon G. Black. The meetings of the Association are held at the Engineers' Club Auditorium.

San Diego Association

The San Diego Association of Members of the American Society of Civil Engineers was organized on February 5th, 1915, and officers have been elected, as follows: President, George Butler; Vice-President, Willis J. Dean; and Secretary-Treasurer, J. R. Comly.

At its meeting of September 20th, 1915, the Board of Direction considered and approved the proposed Constitution of the San Diego Association of Members of the American Society of Civil Engineers.

Seattle Association

The Seattle Association of Members of the American Society of Civil Engineers was organized on June 30th, 1913. At its meeting of January 25th, 1915, the following officers were elected for the ensuing year: President, R. H. Ober; Vice-President, A. S. Downey; and Secretary-Treasurer, Carl H. Reeves.

Southern California Association

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bimonthly meetings, with banquet, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary of the Association, W. K. Barnard, 701 Central Building, Los Angeles, Cal.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

Spokane Association

The proposed Constitution of the Spokane Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on March 4th, 1914. Ulysses B. Hough is President.

Texas Association

The proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers was considered and approved by the Board of Direction of the Society on December 31st, 1913. The headquarters of the Association is Dallas, Tex. John B. Hawley is President.

MINUTES OF MEETINGS OF SPECIAL COMMITTEES TO REPORT UPON ENGINEERING SUBJECTS

Special Committee on Materials for Road Construction

November 4th, 1915.—The meeting was held at the House of the Society. Present, W. W. Crosby (Chairman), A. W. Dean, Nelson P. Lewis, and A. H. Blanchard (Secretary).

The minutes of the meeting of October 23d, 1915, were read and approved.

A resolution was adopted to the effect that the permission of the Board of Direction be obtained to hold a Special Meeting of the Society on Friday, January 21st, 1916, and that the business at that meeting be limited to a discussion of the Committee's Semi-Final Report which is to be submitted to the Annual Meeting on January 19th, 1916.

The sections of the tentative draft of the 1916 Report covering specific conclusions relative to stone block pavements and wood block pavements, methods of testing non-bituminous highway materials, and data forms, were tentatively adopted.

On motion, duly seconded, it was decided that the draft of the 1916 Report, as tentatively adopted at the meetings of the Committee on October 23d and November 4th, 1915, should be adopted as the report of the Committee.

Special Committee on Concrete and Reinforced Concrete

October 22d, 1915.—The meeting was held at the House of the Society. Present, J. R. Worcester (Chairman), William K. Hatt, Olaf Hoff, A. N. Talbot, and Richard L. Humphrey (Secretary). Representatives from the American Society for Testing Materials, the American Railway Engineering Association, and the American Concrete Institute were also present.

After brief discussion by Messrs. Worcester, Hoff, Humphrey, and Talbot, the motion of the Secretary that the Committee try to complete its work with a view to disbanding on June 1st, 1916, was adopted unanimously.

On motion, the budget prepared by the Secretary to cover the expenses of the Committee until July 1st, 1916, was referred to the Committee on Ways and Means, with a request to provide for the expenses of the Committee until June 1st, 1916.

The Secretary moved that a Recommended Practice for Concrete and Reinforced Concrete be prepared, embodying all recommendations of the Committee, including revisions, to be accompanied with a preface giving a general statement of the work of the Committee. This was amended by substituting a brief preface for Chapter I of the present report, and the motion, as amended, was adopted unanimously.

Various portions of the report were then considered in detail, special attention being devoted to the subjects of aggregate, reinforcement, and flat slab design, but final action on these matters was deferred until a later meeting.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Electrical Engineers, 33 West Thirty-ninth Street, New York City.

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 176 Mansfield Street, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of. Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Detroit Engineering Society, 46 Grand River Avenue, West, Detroit, Mich.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, 6 West Eager Street, Baltimore, Md.

Engineers' Club of Kansas City, E. B. Murray, Secretary, 920 Walnut Street, Kansas City, Mo.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Club of Trenton, Trent Theatre Building, 12 North Warren Street, Trenton, N. J.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

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Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, State Museum Building, Chartres and St. Ann Streets, New Orleans, La.

Memphis Engineers' Club, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Oregon Society of Civil Engineers, Portland, Ore.

Pacific Northwest Society of Engineers, 312 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

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Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

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DONATIONS*

PRACTICAL TRACK WORK.

By Kenneth L. Van Auken. Cloth, 8 x 5¼ in., illus., 216 pp. Chicago, Railway Educational Press, 1915. $1.50.

Most of the material contained in this work is original, it is said, the information having been obtained by the author from his experience as a laborer and foreman engaged in track work. Although descriptions of engineering design and maintenance have been omitted, it is said, as having no interest for the practical trackman for whom the book is intended, the young engineer in charge of track work will find much that will prove of benefit to him, such as organization of labor gangs, the actual steps, illustrated by diagrams, to be taken in the construction of tracks and switches, etc. At the end of the book, the author has included a glossary of words referring specifically to track work, as well as tables of temperature expansion for laying rails, dimensions to be used for the frog board for different angle frogs, ordinates for use in curving rails for curves of various degrees, widening of curves and guard rail distances, theoretical and practical switch leads, etc., etc. The Contents are: Labor and Organization; Track Materials and Tools; Spiking, Bolting, Cutting, and Curving Rails, etc.; Constructing Track on a New Line; Double Tracking; Relaying Track; Construction of Turnouts, Ladder Tracks, and Crossovers; Slip Switches; Surfacing New Track; Appendix: Miscellaneous; Glossary of Track Terms; Tables; Index.

RAILWAY MAINTENANCE ENGINEERING

With Notes on Construction. By William H. Sellew. Cloth, 7½ x 5¼ in., illus., 19 + 360 pp. New York, D. Van Nostrand Company, 1915. $2.50.

Railway development in the United States has reached a stage, the author states, where it is intensive rather than extensive, and the young engineer of the present day will be more concerned with the improvement of existing lines than the construction of new ones. The subject-matter of this book, which has been prepared, it is said, from notes used by the author in his classes in Railway Engineering at the University of Michigan, is confined, therefore, principally to maintenance, with a few notes on construction. The arrangement of the chapters follows the classification of investment accounts of the Interstate Commerce Commission. Such subjects as Major Bridges and Yards and Terminals have not been discussed, it is said, because they are beyond the scope of this book and are fully treated of elsewhere, and very few cost data have been included for the same reason. The subject of Signaling, as given herein, is intended only, it is stated, to impart a general knowledge of such work. The book has been written, it is said, from the viewpoint of the engineering student, but the author hopes that sufficient matter of an advanced character has been included to make it of value to the practising engineer. Numerous illustrations and drawings are given, and at the end of each chapter is a bibliography of subjects discussed in that chapter. The Chapter headings are: Engineering; Land; Grading; Bridges, Trestles and Culverts; Ties; Rails; Other Track Material; Ballast; Maintaining Track and Right of Way; Station and Roadway Buildings; Water Stations; Fuel Stations; Shops and Engine Houses; Icing Stations; Signals and Interlockers; Index.

THE CONSTRUCTION OF THE PANAMA CANAL.

By William L. Sibert and John F. Stevens, Members, Am. Soc. C. E. Cloth, 8 x 5¼ in., illus., 10 + 339 pp. New York and London, D. Appleton and Company, 1915. $2.00.

In the preparation of the subject-matter of this book, Mr. Stevens, it is stated, wrote and is responsible for Chapters II to VI, inclusive, and that the Introductory Chapter and Chapters VII to XX, inclusive, were written by Gen. Sibert. Mr. Stevens' contribution, Chapters II to VI, inclusive, covers, it is said, the operations in connection with the building of the Panama Canal prior to the spring of 1907, which is known as the "preparatory period", the period of organization and plant preparation. This period also covers Mr. Stevens' connection with the work as Chief Engineer, and he describes briefly and concisely the reasons for the adoption of the lock type, the sanitary work done in the Canal Zone, the reconstruction of the

* Unless otherwise specified, books in this list have been donated by the publishers.

Panama Railroad, development of working plans, assemblage of plant and labor, housing, feeding, etc. Chapters VII to XX, inclusive, the portion written by Gen. Sibert, covers the work on the Canal from the spring of 1907, to April, 1914, known as the "construction period". Chapters VIII to XI, inclusive, contain a short description of the adopted project, the changes made in that project during construction, with the reasons for such changes, and an outline of the designs for permanent buildings and locks. Chapters XII to XIX relate to the construction work on the Canal, and Chapter XX includes a description of the operation of the Canal, locks, etc., costs of the work being given in Chapter XXI. The Contents are: Introduction; Preparatory Period: 1904 to March, 1907: Sea-Level *versus* Lock Type of Canal; The Reconstruction of the Panama Railroad; Prosecution of the Work; Development of Working Plans; The Housing and Feeding of the Force; The Assemblage and Management of the Force. Construction Period: March, 1907, to April, 1914: The Adopted Project; Changes in the Adopted Project; Changes in Dimensions of Parts of Canal; Designs for Permanent Buildings and Locks; Construction from Colon to Gatun; Excavation and Concrete Work at Gatun; Construction of Gatun Dam; Gatun Lake; Construction from Gatun to Pedro Miguel-Culebra Cut; South End of Culebra Cut to the Pacific Ocean; Municipal Engineering; Shops and Terminal Facilities; Operation of Panama Canal; The Work and Its Cost; Index.

WATER POWER ENGINEERING:

The Theory, Investigation, and Development of Water Powers. By Daniel W. Mead, M. Am. Soc. C. E. Second Edition. Cloth, $9\frac{1}{4}$ x $6\frac{1}{2}$ in., illus., 17 + 843 pp. New York, McGraw-Hill Book Co., 1915. $5.00.

The first edition of this work was published in 1908 and, as many changes have been found necessary to bring the text in accord with the best modern practice, it is stated that various additions, omissions, and re-arrangements have been made in this edition, in order that the subject may be treated more fully and in a more logical order. In the preface to the first edition, the author states that he has endeavored to consider briefly the design, construction, and cost of a water power plant, together with the controlling factors which go to make up a successful development, such as, the adequacy of the supply, the head and power available and their probable variation, the plan for development, the cost of operation, the advisability of the investment, etc. In this edition, the chapters on Hydraulics, Rainfall, and Run-Off, have been omitted, it is said, as not sufficiently complete for the author's purpose, and, in Chapter VI, the discussion of the hydrograph as a basis of power development has been somewhat extended. In Chapters XI and XIII the treatment of the two systems of graphical turbine analysis has been simplified, it is stated, and the author hopes that these methods may now be used more extensively among water power engineers. No attempt has been made, it is said, to discuss in detail the subject of turbine design, except as it is essential to a fundamental knowledge of water power engineering. At the end of each chapter a bibliography of the subject discussed in that chapter, is included. The Chapter Headings are: Introduction; Power; The Load; The Flow of Streams; The Measurement of Stream Flow; A Study of the Power of a Stream as Affected by Flow; Pondage and Storage; Study of the Power of a Stream as Affected by Head; Water Wheels; Turbine Details and Appurtenances; Hydraulics of the Turbine; Turbine Testing; Turbine Analysis and Selection; Speed Regulation of Turbine Water Wheels; The Water Wheel Governor; Arrangement of the Reaction Wheel; Selection of Machinery and Design of Plant; Examples of Water Power Plants; The Relation of Dam and Power Station; Principles of Construction of Dams; Appendages to Dams; Cost of Power Plants and Power; Financial and Commercial Considerations; The Consideration of Water Power Projects; Appendices: Miscellaneous Tables; Test Data of Turbine Water Wheels; Coefficients for Weirs of Various Shapes; Index.

IRRIGATION PRACTICE AND ENGINEERING:

Volume II, Conveyance of Water: General Considerations and Features Pertaining to Irrigation Systems; Conveyance of Water in Canals, Tunnels, Flumes, and Pipes. By B. A. Etcheverry, Assoc. M. Am. Soc. C. E. Cloth, $9\frac{1}{2}$ x $6\frac{1}{4}$ in., illus., 18 + 364 pp. New York and London, McGraw-Hill Book Company, Inc., 1915. $3.50.

This work, of which Volume I, on the Use of Irrigation Water, served as the introductory volume, is devoted, it is said, to a presentation of the fundamental principles and problems of irrigation engineering. Although Volume II is intended for the use of teachers and students as a textbook, sufficient descriptive matter and cost data have been added, it is stated, to make it of value as a reference book for engineers engaged in the construction and operation of irrigation systems. After

a discussion in Chapters I to IV, inclusive, of the general physical features and character of irrigation systems, surveys, hydraulic formulas, and the silt problem, the author devotes the remainder of the volume, as stated in the secondary title, to descriptions of the construction and operation of canals, tunnels, flumes, siphons, and pipes, as conveyors of water for irrigation systems. Discussions of dams for the development of storage, and of high masonry dams for the diversion of water, have been omitted, it is stated, because of the many excellent books on the subject, but much space has been given, it is said, to a rather complete consideration of low dams for diversion weirs. Many drawings and photographs of constructed works are included, and, at the end of each chapter, tabulated references are given to other works pertaining to the subject discussed in that chapter. The subject-matter was prepared in part as a course in Irrigation Engineering presented at the University of California, having been based, it is said, on the author's personal experience and on his observations on the work of other engineers, and it is believed that the opinions and principles presented herein are in accord with correct theory and good practice. The Contents are: General Features and Preliminary Investigations to Determine the General Feasibility of an Irrigation Project; Procedure in the Planning and Location of an Irrigation System; Hydraulic Formulas Specially Applicable to Computations of Irrigation Canals and Structures; Silt Problems in the Design of Irrigation Systems; Conveyance Losses in Canals; The Design of Canal Cross-Sections; Canal Linings and the Prevention of Seepage Losses; Tunnels, Concrete Retaining Wall Canal Sections, Bench Flumes; Flumes; Pipes and Inverted Siphons; Index.

A TEXT-BOOK ON WELDING AND CUTTING METALS

By the Oxy-acetylene Process. By C. H. Burrows. Third Edition, Revised. Cloth, 8¾ x 6 in., illus., 7 + 134 pp. Minneapolis, Minn., Vulcan Process Co., 1915. $1.50.

The purpose of this book, as stated in the preface, is to supply, to mechanics and autogenous workers, short, clear, and practical instructions on the subject of oxy-acetylene weldings. This process has become popular since its first application to commercial uses in 1903, due, it is said, to the ease and economy with which its intense heat is applied by contractors, railroads, etc., and in shipyards, machine shops, foundries, boiler shops, garages, etc., to the welding and cutting of metals. Much that is technical has been omitted, it is stated, as being useless to the practical man who seeks only enough theory to master thoroughly the performance of his duties. The chapters on Chemistry, Physics, and Metals are of the most elementary character, it is said, but are sufficiently explicit to furnish the welder with a thorough working knowledge of these subjects. The following chapters are devoted to descriptions of generators, torches, operating plants, etc., with instructions for their construction and use, and some cost data are included. The Contents are: The Use of the Oxy-Acetylene Flame; Chemistry; Physics; Metals and Their Properties; Acetylene Generators; Oxy-Acetylene Welding and Cutting Torches; Regulators and Indicators; The Vulcan Automatic Acetylene Generator; Operating Plants; Welding Rods and Fluxes; Welding; Cutting; Boiler and Sheet Metal Work; Carbon Burning; Useful Tables and Information; Index.

GENERAL SPECIFICATIONS FOR CONCRETE WORK

As Applied to Building Construction. By Wilbur J. Watson, M. Am. Soc. C. E. Second Edition. Paper, 11 x 8½ in., illus., 56 pp. New York, McGraw-Hill Book Company, Inc., 1915. $1.00. (Donated by the Publishers and the Author.)

The first edition of this work was issued in 1908. Since that time much progress has been made in reinforced concrete design and construction, which, it is stated, has necessitated many radical changes in this volume, in order to bring the subject-matter up to date. The text has been re-arranged, it is said, and considerable that is new has been added. The author, it is said, has endeavored in this edition to conform as far as practicable with the requirements of ordinances governing the design and construction of concrete work now being adopted by many of the large cities, Section XV, on flat-slab floors, being based, it is stated, on the Chicago Building Code modified slightly to meet the views of the author. All contract provisions have been eliminated in this edition, it is said, and the notation used in Section IV has been changed to conform more closely to that adopted by the Joint Committee on Concrete and Reinforced Concrete. The author hopes that with the conservative design and good materials, such as are supposed to be required by these specifications, and with careful supervision and inspection, practically all failures and accidents in structural concrete work will be eliminated. The Contents are: Classes, Definitions, General Provisions and Uses; General Rules for Computing

and Designing; Working Unit Stresses; Formulas; Quality of Materials for Concrete Work; Proportioning, Mixing and Placing Concrete; Requirements for Placing Reinforcing Steel, Inserts, etc.; Placing Concrete in Cold Weather; Forms and Centers; Surface Finish; Waterproofing; Reinforced Steel Construction; Cast Stone and Blocks; Concrete Piling; Flat-Slab Types of Floor Construction; Floor Finish, etc.; Inspection and Tests; General Provisions; Designing Tables and Data.

ELEMENTS OF HIGHWAY ENGINEERING.

By Arthur H. Blanchard, M. Am. Soc. C. E. Cloth, 9¼ x 6¼ in., illus., 12 + 514 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1915. $3.00.

The text of this book, the preface states, consists of original material, together with material from the author's comprehensive text and reference book, "Text-Book on Highway Engineering", and is intended, it is said, to meet the requirements of engineering students who are taking courses in Highway Engineering aggregating from 1 to 3 hours per week for one-half the collegiate year. Each chapter has been written, it is stated, with the view of emphasizing the fundamental principles of the subject, evolved from past experience and modern practice, such as economics, administration, legislation, materials and methods. Specifications, as such, examples of construction, and detailed cost data have been omitted, it is stated, as not being essential to a broad general knowledge of the science of Highway Engineering. In Appendix I, the author has included a glossary of terms relating to materials and methods of highway construction and maintenance. In Appendices II and III, detailed descriptions are given of methods for determining the physical and chemical properties of bituminous and non-bituminous highway materials, which, it is said, will be found valuable by teachers who wish to elaborate on the subject of essential properties of materials and their determination. The text is fully illustrated and many references to other works on the subject are included. The Contents are: Historical Review; Economics, Administration, Legislation, and Organization; Preliminary Investigations; Surveying, Mapping, and Design; Grading, Drainage and Foundations; Earth and Sand-Clay Roads; Gravel Roads; Broken Stone Roads; Bituminous Materials; Dust Prevention and Bituminous Surfaces; Bituminous Macadam Pavements; Bituminous Concrete Pavements; Sheet Asphalt and Rock Asphalt Pavements; Cement-Concrete Pavements; Wood-Block Pavements; Brick Pavements; Stone Block Pavements; Street Cleaning and Snow Removal; Comparison of Roads and Pavements; Sidewalks, Curbs and Gutters; Highway Structures; Appendix I, Glossary of Terms Applicable to Highway Engineering; Appendix II, Tests for Bituminous Materials; Appendix III, Tests of Non-Bituminous Materials; Index.

THE MODERN CITY AND ITS PROBLEMS.

By Frederic C. Howe. Cloth, 8¼ x 5½ in., 10 + 390 pp. New York, Chicago, Boston, Charles Scribner's Sons, 1915. $1.50.

This book, the preface states, is an analysis of the city from the inside, and is the result of many years of service on the part of the author in the City Council of Cleveland, Ohio, and as a member of various municipal commissions, etc., as well as of an intimate knowledge of many other American cities, of contact with reformers and politicians, of studies of municipal conditions in Germany, England, Austria-Hungary, and Switzerland, and of personal acquaintance with officials of these countries. In the course of analysis the author, it is said, has endeavored to show, by comparison with the municipal activities and powers of European cities, how the American city has lagged behind in the work it should perform for the benefit of the individuals within its limits, how such neglect has resulted in partisanship, tolerance of evil, low wages, irregular employment, and disease, and how such conditions may be corrected by a programme of city service and building, through compulsory co-operation or socialization. The subject-matter, it is stated, reviews the viewpoint of men who are striving for and doing better things animated by a desire to promote municipal achievements and realize the city's possibilities. At the end of the book, the author has included an extensive bibliography of the subject. The Chapter headings are: The City and Civilization; The Ancient City; The Mediæval Town; The Modern City; The American City: Its Success and Its Failures; The City and the State; Municipal Home Rule; The City Charter; Municipal Administration in Germany; Municipal Administration in Great Britain; The City and the Public Service Corporation; Municipal Ownership in America; Municipal Ownership in Europe; City Planning in America; City Planning in Europe; Police, Fire, and Health Protection; The City as a Social Agency; The Housing Problem; Municipal Housing in Europe; Recreation and the Problem of Leisure; The City Budget; New Sources of City Revenue; Conclusion; Bibliography; Index.

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A secondary title states that this book is a complete and reliable annual register of producers, manufacturers, dealers, and consumers connected with the industries mentioned above, and with other industries of interest to buyers and sellers. It is said to contain full lists of products from the raw material to the finished article, and of companies handling such products from the producer to the consumer, and is stated to be indispensable as a buyer's reference and for mailing purposes. The contents are arranged alphabetically by subject, under which are given, in alphabetical order, and sometimes by States and cities, the names and addresses of firms dealing in the various articles, and these are followed, in some cases, by detailed matter, titles of identification, trade names, etc. There is also an alphabetical list of advertisers, including the addresses of their domestic and foreign branches, and an index of contents of 147 pages.

INTEGRATION BY TRIGONOMETRIC AND IMAGINARY SUBSTITUTION.

By Charles O. Gunther, Assoc. Am. Soc. C. E. With an Introduction by J. Burkett Webb. Second Edition, Corrected. Cloth, 8 x 5½ in., illus., 6 + 79 pp. New York, D. Van Nostrand Company, 1915.

The method of integration developed in this book is designed, it is stated, to eliminate the "reduction formulas" and make the student independent of textbooks and tables of integrals. It has been used successfully by the author in his classes, and as it is founded on trigonometric principles, the student, it is said, becomes proficient not only in the integration of differential expressions, but also in the transformation of algebraic expressions into trigonometric and exponential ones. The subject-matter consists of a series of problems in the integration of trigonometric forms, with their answers. This is preceded by an Introduction in which the general principles involved are reviewed, and the book is intended, it is said, for use with textbooks on the subject after the student has become familiar with the simple rules of integration resulting from the reversion of rules for differentiation. The Contents are: Introduction; Trigonometric Differentials; Rationalization by Trigonometric Substitution.

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Automatic Sprinkler Protection. By Gorham Dana. Boston, 1914.

Unit Construction Costs from the New Smelter of the Arizona Copper Co., Ltd. By E. Horton Jones. New York and London, 1914.

American Society for Testing Materials Affiliated with the International Association for Testing Materials. Proceedings of the Eighteenth Annual Meeting, 1915; Vol. 15, Parts 1 and 2. Philadelphia. 1915.

Government of the Canal Zone. By George W. Goethals. Princeton and London, 1915.

Power Heating and Ventilation. By Charles L. Hubbard. Part 1, Steam Power Plants (Second Edition); Part 2, Heating and Ventilating Plants (Second Edition); Part 3, Combined Power and Heating Plants. New York and London, 1915.

SUMMARY OF ACCESSIONS

(From November 2d to December 1st, 1915)

Donations (including 32 duplicates)	413
By purchase	14
Total	427

MEMBERSHIP

(From November 5th, to December 2d, 1915)

ADDITIONS

MEMBERS

Member	Grade	Date of Membership.
BEVAN, LYNNE JOHN. Hydr. Engr., Vielé, Blackwell & Buck, 49 Wall St., Room 601, New York City	Assoc. M. M.	May 3, 1910 Nov. 10, 1915
BLISS, GEORGE HENRY. With Bliss & Perry Mfg. Co., 10 Allen St., Newburyport, Mass.	Assoc. M. M.	Sept. 7, 1904 Sept. 20, 1915
BRUNNIER, HENRY JOHN. Cons. Structural Engr., 312 Sharon Bldg., San Francisco, Cal.	Assoc. M. M.	Jan. 3, 1911 Nov. 10, 1915
BURWELL, ROBERT LEMMON. Asst. Div. Engr., Baltimore Sewerage Comm., Am. Bldg. (Res., 118 West Lafayette Ave.), Baltimore, Md.	Jun. Assoc. M. M.	Dec. 3, 1901 July 1, 1908 Nov. 10, 1915
GRISWOLD, HOWARD CLIFTON. Asst. Insp. Engr., Illinois Steel Co., 424 West Main St., Louisville, Ky.		Nov. 3, 1915
HOGAN, JOHN PHILIP. Div. Engr., Board of Water Supply, City of New York, 100 Barrett Boulevard, Tompkinsville, N. Y.	Jun. Assoc. M. M.	Dec. 6, 1904 Mar. 5, 1912 Nov. 10, 1915
JONES, JONATHAN. Asst. Engr., Bureau of Surveys, 532 City Hall, Philadelphia, Pa.	Assoc. M. M.	Sept. 6, 1910 Nov. 10, 1915
KIRKBRIDE, WALTER HAROLD. Div. Engr., S. P. Co., 1700 H St., Sacramento, Cal.	M.	Nov. 3, 1915
McCRORY, SAMUEL HENRY. Chf., Drainage Investigations, Office of Experiment Station, U. S. Dept. of Agriculture, 6811 Sixth St., Takoma Park, D. C.		June 3, 1915
PHIPPS, THOMAS ELMER. Chf. Engr., Public Service Comm. of Washington, 3444 Florence Court, Seattle, Wash.	Assoc. M. M.	Oct. 3, 1911 Nov. 10, 1915
REEVES, CARL HOWELL. Cons. Engr., 444 Henry Bldg. (Res., 4722 Latona Ave.), Seattle, Wash.	Jun. Assoc. M. M.	May 2, 1905 July 1, 1909 Nov. 10, 1915
RICHMOND, WALDEMAR SPAULDING. Junior Engr., U. S. Lake Survey, Old Custom House, Detroit, Mich.	Assoc. M. M.	Oct. 3, 1911 Nov. 10, 1915
RITTER, HOMER PETER. Asst., U. S. Coast and Geodetic Survey; Member, Mississippi River Comm., Washington, D. C.		Nov. 3, 1915
SCOTT, CHARLES BRUCE. Asst. State Highway Commr., Care, State Highway Comm., Richmond, Va.	Assoc. M. M.	Feb. 3, 1904 Nov. 10, 1915
STEPHENSON, FRANK HENRY. Engr. of Design, Bureau of Filtration, City Hall, Cleveland, Ohio	Assoc. M. M.	Jan. 8, 1908 Nov. 10, 1915

ASSOCIATE MEMBERS

		Date of Membership.
AGUIRRE, FRANK MONZON. Cons. Engr.; Asst. Gen. Mgr. and Chf. Engr., Cienfuegos Water and Sewerage Systems, Santa Clara 252, Cienfuegos, Cuba		Nov. 3, 1915
ALLEN, CHARLES LYLE. Asst. Engr., C. & O. Ry., Ronceverte, W. Va		Nov. 3, 1915
CARPENTER, SELLECK TOWNSEND MANN. Chf. Engr. and Estimator, B. I. Crooker, 250 Pearl St., Buffalo, N. Y.		Nov. 3, 1915
COANE, HENRY EDWARD. Civ., Hydr., and Min. Engr., 70 Queen St., Melbourne, Victoria, Australia		Aug. 31, 1915
COMLY, HARRY SEYKORA. Asst. Div. Engr., California Highway Comm., Div. 2, Dunsmuir, Cal		Nov. 3, 1915
DISHER, QUENTIN FRASER. Woodmere, N. Y		Nov. 3, 1915
DONNELLY, JOSEPH FRANCIS SINNOTT. Chf. Draftsman, Cuba R. R., Camaguey, Cuba		Aug. 31, 1915
DORT, JOSEPH CUMMINGS. 230 Newbury St., Boston, Mass.		Nov. 3, 1915
EGENHOFF, ROWLAND LEONARD. 3419 Andover St., Oakland, Cal		Nov. 3, 1915
FINCH, JAMES KIP. Instr. in Civ. Eng., Columbia Univ. (Res., 126 Claremont Ave.), New York City	Jun. Assoc. M.	June 4, 1907 Nov. 3, 1915
FRASER, EDWIN ARCHIBALD. Care, Concrete Steel Co., 42 Broadway, New York City		Nov. 3, 1915
GARDINER, JOHN DEBARTH WALBACH. Engr. and Sales Mgr., Minwax Co., Inc., 18 East 41st St., New York City		Nov. 3, 1915
GASS, HOWARD RAY. 801 East High St., Jefferson City, Mo.		Nov. 3, 1915
GLASS, WILLIAM ASA. Res. Engr., Fla. East Coast R. R., Marathon, Fla		Nov. 3, 1915
GUISSINGER, JOHN ADAMS. Care, Cleburne Hotel, Helena, Mont	Jun. Assoc. M.	Sept. 5, 1911 June 3, 1915
HAMILTON, JOHN ANDREW. Care, The Birmingham Water Works Co., 2114 First Ave., Birmingham, Ala		Nov. 3, 1915
HENRY, EARLE UNDERWOOD. City Engr., Orange, Tex		Nov. 3, 1915
HOGE, EDWARD AUGUSTUS CLYDE. Chf. Engr., Keystone Fireproofing Co., 47 West 34th St., Room 1020, New York City		Nov. 3, 1915
JACOBS, SELWYN SIMON. Supt., James Stewart & Co., Inc., 1858 West 14th St., Chicago, Ill		Nov. 3, 1915
JACOBY, HURLBUT SMITH. Asst. Chf. Engr., The Samuel Austin & Son Co., 1849 Lampson Rd., Cleveland, Ohio.		Nov. 3, 1915
KASANO, YEIZO. With Imperial Govt. Rys., Kobe, Japan		Aug. 31, 1915
MOORE, WILLIAM GERALD. Asst. Engr., B. & O. R. R., 305 Goodwood Rd., Roland Park, Md		Nov. 3, 1915
NAWN, HUGH. Asst. to the Pres., The Hugh Nawn Contr. Co., 188 Seaver St., Roxbury, Mass	Jun. Assoc. M.	Jan. 31, 1911 Nov. 3, 1915
POIROT, ALOIS PHILLIP. Belleville, Ill		Aug. 31, 1915

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
RICHEY, CHARLES WILSON. Master Carpenter, Pittsburgh Div., P. R. R., 814 La Schall St., Pittsburgh, Pa....		Nov. 3, 1915
RIDGEWAY, GEORGE ALLEN. Highway Engr., Boone County, Columbia, Mo....		Nov. 3, 1915
SCHARFF, MAURICE ROOS. Prin. Asst. Engr., Morris Knowles, 2541 Oliver Bldg., Pittsburgh, Pa....	Jun. Assoc. M.	Feb. 6, 1912 Nov. 3, 1915
SWICKARD, JAMES BLAINE. 641 South 11th St., San José, Cal....	Jun. Assoc. M.	April 30, 1912 Nov. 3, 1915
TERRY, JAMES CAD. Pres., American & Brazilian Eng. Co., Caixa Postal 64, Curityba, Parana, Brazil....		Aug. 31, 1915
TYLER, RICHARD GAINES. City Engr., Paris, Tex....	Jun. Assoc. M.	Oct. 6, 1908 Nov. 3, 1915
WHITCRAFT, LEWIS NORRIS. Chf. Engr., Hydrated Lime Bureau of the National Lime Mfrs. Assoc., 439 Oliver Bldg., Pittsburgh, Pa....		Nov. 3, 1915

JUNIORS

BACON, ETHELBERT. Asst. Engr., Clyde Potts, 30 Church St., New York City....	Aug. 31. 1915
BARBER, CHARLES WIGHTMAN. First Field Asst., Highway Div., Engr. Dept., 3428 Brown St., N. W., Washington, D. C....	Nov. 3, 1915
FROISETH, RICHARD EUGENE. 855 Sherman Ave., Salt Lake City, Utah....	June 3, 1915
HIGGINS, THOMAS CAROL. 15 Des Isle Ave., Bar Harbor, Me..	Nov. 3, 1915
ROSENFELD, JAMES ROY. Care, Chester & Fleming, Union Bank Bldg., Pittsburgh, Pa....	Nov. 3, 1915
ROYAL, JOSEPH NELSON. Care, Barrett Mfg. Co., 17 Battery Pl., New York City....	Nov. 3, 1915
SACKS, SAMUEL ISAIAH. Draftsman, Dept. of City Transit, 1755 North 31st St., Philadelphia, Pa....	Nov. 3, 1915
SEELYE, SETH HENESS. Miami Conservancy Dist., Dayton, Ohio....	Nov. 3, 1915
WILLIAMS, ALAN FRANK. 419 West Lime Ave., Monrovia, Cal....	Nov. 3, 1915

CHANGES OF ADDRESS

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ANDERSON, GEORGE GRAY. Cons. Engr., 436 Consolidated Realty Bldg., Los Angeles, Cal.

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BANDEL, JOHN MARTIN. Asst. Engr., Sewerage Comm. of Baltimore, 37 East 25th St., Baltimore, Md.

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BODYCOMB, WALTER CHRISTMAS. Care, Westinghouse, Church, Kerr & Co., National Stock Yards, Ill.

BRIGGS, HARRY ALSON. Arcadia, Fla.

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BROWNLEE, JAMES LAWRENCE. Ada, Ohio.

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CARHART, FRANK MILTON. Sherman House, Boise, Idaho.

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COLGAN, CHARLES JUDSON. Grade Crossing Engr., The Public Service Comm., R. F. D. No. 1, Medical Lake, Wash.

COLMAN, WILLIAM TUCKER. Engr. in Chg., Marine Dept., Lincoln Park, 612 Wrightwood Ave., Chicago, Ill.

CONNER, CARLTON NUDD. Junior Engr., U. S. A., U. S. Engr. Office, P. O. Box 716, Cincinnati, Ohio.

CRANE, ERNEST BUCHANAN. Asst. Engr., C., M. & St. P. Ry., 63 East Adams St., Chicago, Ill.

CUNNINGHAM, JOHN GEORGE LAWRENCE. Cons. Engr., Deer Lodge, Mont.

CUNNINGHAM, MAX LEE. Asst. Engr., Benham Eng. Co., 13th Floor, Colcord Bldg., Oklahoma, Okla.

DAVENPORT, JAMES WATSON. Hattiesburg, Miss.

DIESEM, HARRY CUSTER. Irrig. Engr., Irrig. Investigation, Office of Public Roads, and Rural Engr., U. S. Dept. of Agriculture, 301 Tramway Bldg., Denver, Colo.

DRAKE, HENRY PHILKINS. Asst. Engr., State Dept. of Health, Box 185, Harrisburg, Pa.

FARRINGTON, HAROLD PHILLIPS. 31 Nassau St., New York City.

FOWLER, CHARLES WORTHINGTON. Box 757, San Juan, Porto Rico.

FOWLER, JAMES DUNCAN. Cons. Engr. (Koch & Fowler), Box 82, Holton, Kans.

GOODMAN, CHARLES. Asst. Engr., Board of Water Supply, City of New York, 661 West 180th St., New York City.

GREENE, RUSSELL DE COSTA. 105 Thomas St., Cranford, N. J.

GREENLEAF, FRED BACON. Contr. (J. A. Greenleaf & Sons), 121 Spring St., Auburn, Me.

GUSTIN, R PROSPER. 136 Urban St., Mount Vernon, N. Y.

HALL, JULIUS REED. Cons. Engr., 105 West Monroe St., Chicago, Ill.

HART, RICHARD AMBROSE. Superv. Drainage Engr., U. S. Dept. of Agriculture, Federal Bldg., Berkeley, Cal.

HARTRIDGE, EARLE MENELAS. 1050 Eddy St., San Francisco, Cal.

ASSOCIATE MEMBERS (*Continued*)

HESLOP, DERWENT GORDON. Locating Engr., Chilaw-Puttalam Ry., Chilaw, N. W. P., Ceylon.

HICKOK, CLIFTON EWING. 344 East 54th St., Portland, Ore.

HIGGINSON, JONATHAN YATES. Care, Henry Steers, 17 Battery Pl., New York City.

HOWARD, OLIVER ZELL. Cons. Engr., 851 West 181st St., New York City.

HOWES, CYRUS PIERCE. Care, Ernest W. Wiggin, 506 Cutler Bldg., New Haven, Conn.

JARVIS, CLARENCE SYLVESTER. 376 Fifth Ave., Salt Lake City, Utah.

JENKINS, JAMES EDGAR. Supt., The Cook Constr. Co., McGill Bldg., Montreal, Que., Canada (Res., 644 Riverside Drive, New York City).

JONSON, ERNST FREDRIK. Engr. Insp., Board of Water Supply, 49 Lafayette St., New York City.

KAYSER, EDWARD MATHEW. Engr. and Supt., Mason, Hilton & Co., 326 Park Ave., Paterson, N. J.

KERLINGER, WILLIAM MURRAY. Box 49, Stratford, Cal.

KIRKPATRICK, GEORGE DALLAS DIXON. Asst. Superv. of Surveys, Gen. Land Office, 1367 Yale Ave., Salt Lake City, Utah.

LEONARD, OLIVER YEATON. Southern Hotel, Columbus, Ohio.

LOEWE, DAVID LUDWIG. Care, E. H. Erlanger, 60 Wall St., Room 915, New York City.

LORAH, WALTER LAWRENCE. Sonestown, Pa.

LYNCH, ALEXANDER SYDNEY. 57 Holmes St., West Haven, Conn.

McRAE, HENRY CLINTON. Dist. Engr., International Joint Comm., 3413 Piedmont Ave., Baltimore, Md.

MAGRUDER, FRANK CECIL. Scottsbluff, Nebr.

MAHONE, WILLIAM, JR. Care, So. Ry., Box 33, Danville, Va.

MARTIN, CHARLES CHRISTOPHER. Care, L. A. C. Boulay Co., 1250 Nicholas Bldg., Toledo, Ohio.

MOLINA, VICENTE. Engr., Constr., San Luis Bridge, for Snare & Triest Co., Apartado No. 271, Matanzas, Cuba.

NOLAN, THOMAS BREW, JR. Engr., Westinghouse, Church, Kerr & Co., Matanzas, Cuba (Res., The Hawarden, Washington, D. C.).

OSBORNE, HERBERT. Box 209, Penticton, B. C., Canada.

OVEROCKER, DANIEL WILLETS. Asst. Engr., Barge Canal Office, Lyon Blk., Albany, N. Y.

PIERCE, CHARLES HENRY. Dist. Engr., U. S. Geological Survey, Custom House Bldg., Boston, Mass.

PINNER, GUY. 130 West Commerce St., San Antonio, Tex.

PRATT, WALTER MITCHELL. Mgr., States Eng. Co., 514 Free Press Bldg., Detroit, Mich.

RABINOWITZ, LOUIS. 1206 Fifth Ave., New York City.

RAY, FRANK OLIVER. Engr., The R. Hardesty Mfg. Co., Broadmoor, Colorado Springs, Colo.

ROBINSON, HARRY. Supt., Atlantic Const. & Supply Co., Christiania Apartments, Atlantic City, N. J.

ASSOCIATE MEMBERS (*Continued*)

SEYMOUR, HORATIO. Section Engr., Public Service Comm., First Dist., New York State, Route 16, Section No. 2, Bronxville, N. Y.

SHOEMAKER, JOHN EARL. Asst. Engr., Port of Seattle, 2707 Thirty-third Ave., South, Seattle, Wash.

SKINNER, BENJAMIN BAKER. 162 West 80th St., New York City.

SLOAN, NORTON QUINCY. Corner, First and Orange Sts., Newark, N. J.

SYKES, GEORGE. 40 West 32d St., New York City (Res., Bayside, N. Y.).

TAYLOR, HENRY. Asst. Engr., Erecting Dept., Am. Bridge Co. of New York, 1712 Widener Bldg., Philadelphia, Pa.

TOWNSEND, FRANK THORN. Asst. to Chf. Engr., The Dunn Wire Cut Lug Brick Co., 786 Main St., Conneaut, Ohio.

VILLA, MIGUEL. Engr., Havana Sewer System, Obras Publicas, Cuba 24, Havana, Cuba.

WANZER, JAMES OLIN. Care, California Highway Comm., Div. No. 3, Forum Bldg., Sacramento, Cal.

WENIGE, ARTHUR EMIL. 340 Columbia Ave., Woodhaven, N. Y.

WHEAT, GEORGE NEVILLE. 709 Hitt St., Columbia, Mo.

WICKHAM, HENRY ROGERS. Asst. Engr., Public Service Comm., First Dist., 8 Fourth Ave., Brooklyn, N. Y.

WICKLINE, GEORGE GROVER. Care, County Engr., Dallas County, Dallas, Tex.

WILCOCK, FREDERICK. Asst. Engr., Public Service Comm., 154 Nassau St., New York City (Res., 859 Fifty-first St., Brooklyn, N. Y.).

WOODHOUSE, SIDNEY JAMES. Engr. and Contr. (Medley, Woodhouse & Garayta), Maceo No. 29 (Altos), Sagua la Grande, Cuba.

WRIGHT, THOMAS TEMPLE. Care, Van Ormer Coal & Coke Co., Fallen Timber, Pa.

ASSOCIATES

LIEBMANN, ALFRED. 525 Park Ave., New York City.

MORAN, ROBERT BRECK. 208 Union League Bldg., Los Angeles, Cal.

POLK, WILLIAM ANDERSON. Care, The Commercial Attaché, Dept. of Commerce, U. S., No. 36, Ave. de l'Opera, Paris, France.

TOWNSEND, FREDERICK EUGENE. Archt., 103 Park Ave., New York City.

JUNIORS

BOWLUS, FRED DREXEL. Deputy County Surv., Los Angeles County, Box 354, San Fernando, Cal.

BRINGHURST, HORACE MORTON. Draftsman, McClintic-Marshall Const. Co., 568 Coal St., Wilkinsburg, Pa.

BRYSON, CARLYLE HUGO. 2104 West Boulevard, Los Angeles, Cal.

BURR, GEORGE LINDSLEY. 6 Concord Ave., White Plains, N. Y.

CAMPBELL, PAUL CALDWELL. 400 Interstate Bldg., Kansas City, Mo.

COLLINS, GEORGE MILES. Asst. Engr., City of Honolulu, Hotel Courtland, Honolulu, Hawaii.

CORBIN, HORACE KELLOGG. Engr. and Contr., 30 Church St., New York City.

CROLL, HERBERT GREISS. Designer with City of Toronto, 189 Fulton Ave., Toronto, Ont., Canada.

JUNIORS (*Continued*)

CURTIS, HAROLD EDWIN. Care, Michigan Limestone & Chemical Co., Rogers City, Mich.

CYKLER, EMIL FRANK. Care, Lord & Young Eng. Co., Ltd., Pantheon Bldg.. Honolulu, Hawaii.

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FROST, EDWARD MURRAY. Care, The Ozark Smelting & Min. Co., Coffeyville, Kans.

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YOUMANS, GEORGE LELAND. Care, D. E. Austin, Brodhead, Wis.

RESIGNATIONS

MEMBERS	Date of Resignation.
PASCHKE, THEODORE	Nov. 9, 1915

DEATHS

ALLEN, WILLIAM FREDERICK. Elected Associate Member, January 3d, 1900; died November 9th, 1915.

SHEDD, JOEL HERBERT. Elected Member, September 15th, 1869; died November 27th, 1915.

WENTWORTH, CHARLES CHANCELLOR. Elected Member, April 4th, 1888; died November 11th, 1915.

Total Membership of the Society, December 2d, 1915, 7 915.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(November 2d to December 1st, 1915)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., St. Louis, Mo., 30c.
(2) *Proceedings*, Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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Vol. XLI. DECEMBER, 1915. No. 10.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE FAILURE AND RIGHTING OF A MILLION-BUSHEL GRAIN ELEVATOR

By Alexander Allaire, M. Am. Soc. C. E.

To be Presented February 2d, 1916.

Synopsis.

The object of this paper is to present to the attention of the members of the Society the history of a rather unusual engineering feat. The size of the building to be straightened, the angle to which it had tipped, and the weight to be handled, all combined to make the work unique.

The restoration of the elevator to a working condition may be divided into four parts:

1.—Making safe the foundations under the workhouse—a structure resembling a tall office building, resting on a very small base;

2.—Straightening the binhouse, a structure having an area of 15 000 sq. ft.;

3.—Providing this binhouse with a new and adequate foundation; and

4.—The renewal and repair of those portions of the original buildings which had been broken or deranged at the time of the failure.

This paper treats of the first three.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The Canadian Pacific Railway, which serves as the principal outlet from Western Canada, in 1911, found it imperative to provide relief for its Winnipeg Yards, which were yearly becoming less and less able to meet the demands on them. During October, November, and December, Canada's immense wheat-producing provinces of Manitoba, Saskatchewan, and Alberta ship their grain to the United States and Europe *via* the Canadian Pacific Railway to Fort William, where it is loaded on the Lake boats. To relieve the congestion, a cut-off was made, running approximately 3 miles north of Winnipeg, and one of the largest railroad gravity yards in the world was built at North Transcona, 7 miles northeast of the city. As an additional aid to the speedy shipment of the grain, a million-bushel elevator was constructed at the North Transcona Yard. This is to serve for storage during the "peak" periods, and relieve the car shortages by sending empties back West several days earlier than would be possible, should these cars have to go as far as Fort William.

The country in the vicinity of Winnipeg is flat, and the character of the soil is quite uniform throughout the district. First there is a stratum of about 2 ft. of heavy black loam, below which there is reddish gray clay, 5 or 6 ft. in thickness, and generally water-bearing, and this gradually changes to a blue clay extending to a depth of approximately 40 ft. below the surface, where it changes suddenly to white clay interspersed with limestone boulders. Underlying the white clay is found a stratum, averaging about 30 in. thick, of shattered limestone which in turn overlies the limestone rock. At Transcona this rock varied from 53 to 55 ft. below the prairie level.

Usually, the blue clay found at the depth from 7 to 8 ft. below the surface is very firm, and is capable of carrying a load of from 3 to 4 tons per sq. ft. As a result, it has been the practice in the Canadian Central West to use floating foundations. In building the elevator, therefore, this general custom was followed.

The ultimate loading of the clay under the mat was calculated at 3.3 tons per sq. ft. As a matter of precaution, soil-loading tests were made at different points on the site. From the result, it was thought that the customary slight initial settlement might be experienced, but that nothing more serious should occur.

The general layout consists of four units; a workhouse, 70 by 96 ft. and 180 ft. high; a binhouse, 77 by 195 ft. and 102 ft. high; a dryer-

house, 18 by 30 ft. and 60 ft. high; and a boiler-room equipped with two 100-h.p. locomotive boilers. The workhouse rests on a reinforced concrete floor, 30 in. thick, and is a reinforced concrete structure, with brick curtain-walls as the top is approached. The basement of the building is 16 ft. high. In it are the belts for transferring the grain from the cars to the conveyor-boots, and from the binhouse to the workhouse. The ground, or prairie-level, floor is occupied by the cleaning and drying machinery. Above these machines there are fifteen bins, 13 ft. in diameter and 70 ft. high, above which again are several floors carrying the weighing machines, etc. The floor, machinery, and wall loads above the second floor rest on twenty-four interior columns, placed in four rows of six columns each. The interior columns carry a load of about 800 tons each; the exterior ones carry about 500 tons each. Fig. 1 shows the workhouse in plan and elevation.

The binhouse, immediately north of the workhouse, consists of sixty-five circular bins arranged in five rows of thirteen each. These bins or tanks are 14 ft. 4 in. in diameter. The diamond-shaped spaces formed between the circular bins are also used for storage, each having a capacity of about 5 000 bushels. The bins, 92 ft. high, are surmounted by a cupola which houses the top conveyor and trippers, and extends the full length of the structure. With this the tanks are filled with grain. The bin walls are of concrete, 6 in. thick, with the customary reinforcement. Below the 12-in. reinforced concrete slab, which constitutes the bin bottoms, there are four conveyor-belt tunnels, 7 ft. wide, running the full length of the structure. These tunnels are formed by walls 16 in. thick and 7 ft. high, which rest on a 2-ft. mattress of concrete. Transverse to the main tunnel walls there are 15-in. cross-walls approximately 15 ft. from center to center. These are under the bin contacts. The tunnel or cross-walls were not bonded to the floor or bin slabs, and this constituted a considerable hazard during the straightening.

Excavation for the structure had been made to a depth of 12 ft. below the prairie level.

The dryer-house had broad footing courses under the walls. Although it was deemed advisable, later, to underpin this building, this presented no difficulty.

The Canadian Pacific Railway commenced storing grain in the binhouse in September, 1913. Considerable care was taken to regulate

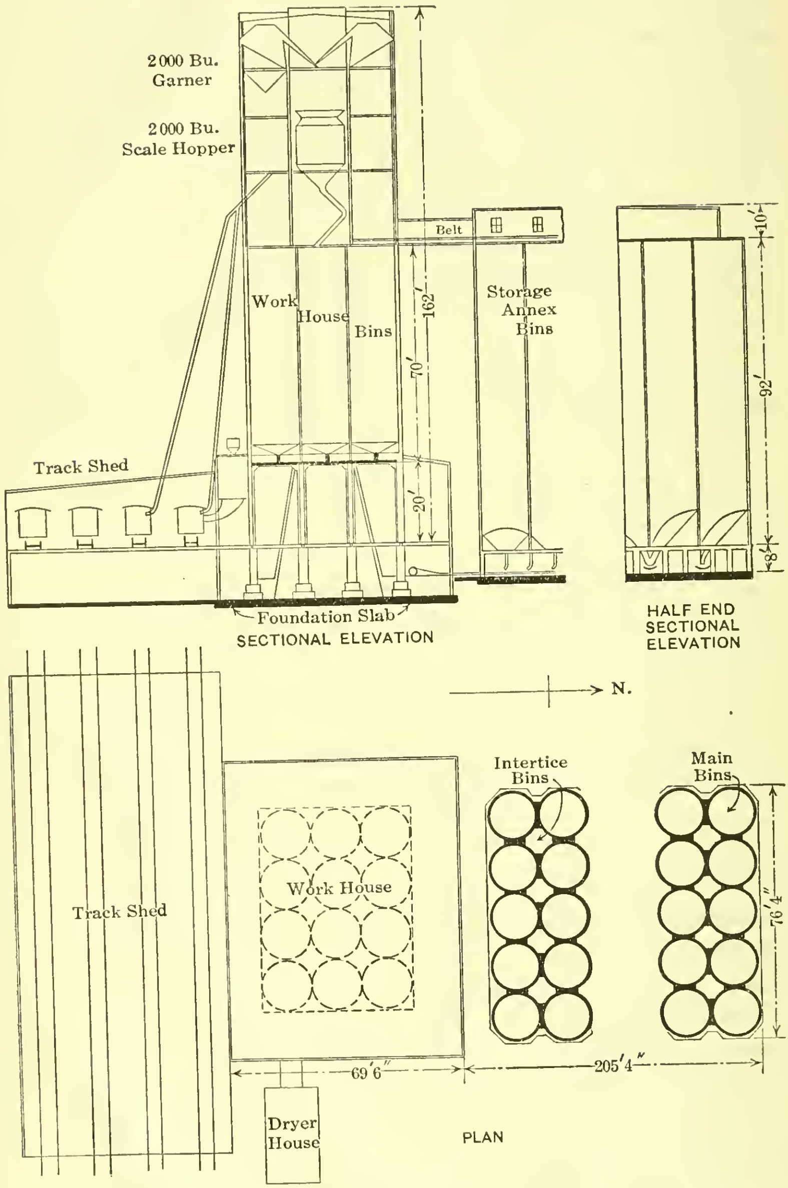
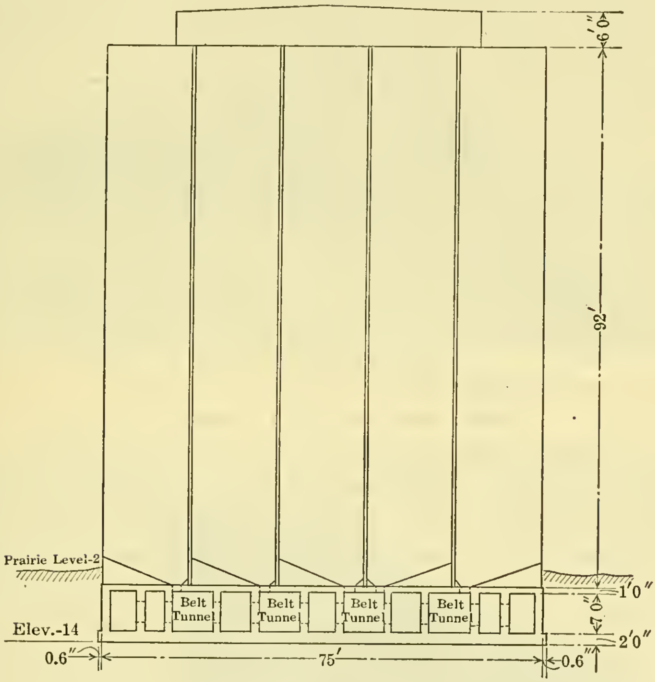

2000 Bu.
Garner
2000 Bu.
Scale Hopper
Belt
Work
House
Bins
162'
70'
20'
Storage
Annex
Bins
10'
92'
8'
Track Shed
Foundation Slab
SECTIONAL ELEVATION
HALF END
SECTIONAL
ELEVATION
N.
Intertice
Bins
Main
Bins
Track Shed
Work House
76'4"
69'6"
205'4"
Dryer
House
PLAN

Fig. 1.

the filling of the different tanks so as to distribute the load uniformly. Settlement began on October 18th. The bins at this time contained about 875 000 bushels of wheat. A vertical sinking of 1 ft. occurred within an hour after the settlement was first noted. This was followed by an inclination toward the west, which increased until, at the end of 24 hours, the binhouse rested at an angle of 26° 53′ from the vertical.



ORIGINAL POSITION
SECTION THROUGH BINS—LOOKING SOUTH

FIG. 2.

An examination showed the east side to be 5 ft. above, and the west side 29 ft. below, the original position; the building was also approximately 4 ft. lower at the north than at the south end. An upheaval of 5 or 6 ft. of the ground surrounding the bins occurred during the settlement. At an angle of 26° 53′ the center of gravity of the loaded structure had moved over almost to the low edge. The upheaval and the compacting of the soil along the west side saved it from completely falling over. Above this, on the west side, 52 ft.

of tank overhung the ground. This overhanging load, due to the grain-filled tanks, was considerable.

A careful examination was made of the structure and, remarkable as it may seem, practically no shear cracks were found. It was decided, therefore, that the first problem was to save the wheat. This was done by tapping the most westerly row of tanks at approximately the ground level and bleeding out the grain upon a belt conveyor parallel with the line of the tanks. When the tanks of this outer row were emptied down to the ground line, holes were tapped in the next row, and the grain was spouted to the belt conveyor, and so on from tank row to tank row. The hazard of this operation will be realized when one considers that it was difficult to calculate what stresses were set up, due to the inclined position of the bins and the change in loading as the tank rows were emptied in succession. Added to this was the menace, at the top of the structure, of the remnants of the cupola, part of which had fallen to the ground during the settlement. To remove the grain below the ground level, a sheeted pit was excavated at the north end. A conveyor leg was placed in this pit, with the discharge end emptying upon the belt parallel with the west side of the bins. The tanks were emptied by spouting the grain to the tunnel belts, which, being reversed, carried it to the conveyor leg. It is worthy of note that, despite the handicap of the dangerous working and the fact that part of the conveying apparatus had to be obtained from Chicago, all the wheat was removed in less than 3 weeks from the time of the failure, and at a cost of less than 1 cent per bushel.

Immediately after the failure an examination was made with boring machines, and it was found that though the rock over the greater area was at a depth of from 52 to 55 ft. below the prairie level, an unusual condition existed along about one-half of the length of the east side of the bins—a ridge of boulders some 12 ft. higher than the rock being encountered. This was the reason for the tipping over, for, as the initial vertical settlement took place, resistance was offered along the east side, probably through compacting the soil above the ridge of boulders. This produced a tendency of the building to cant to the west. The heavy load caused the clay to flow and the structure to settle.

Fortunately, the workhouse was only slightly affected by the movement of the binhouse. Only a few cracks showed in the north shed wall, no settlement taking place in the building proper.

FIG. 3.—VIEW OF ELEVATOR AFTER SETTLEMENT.

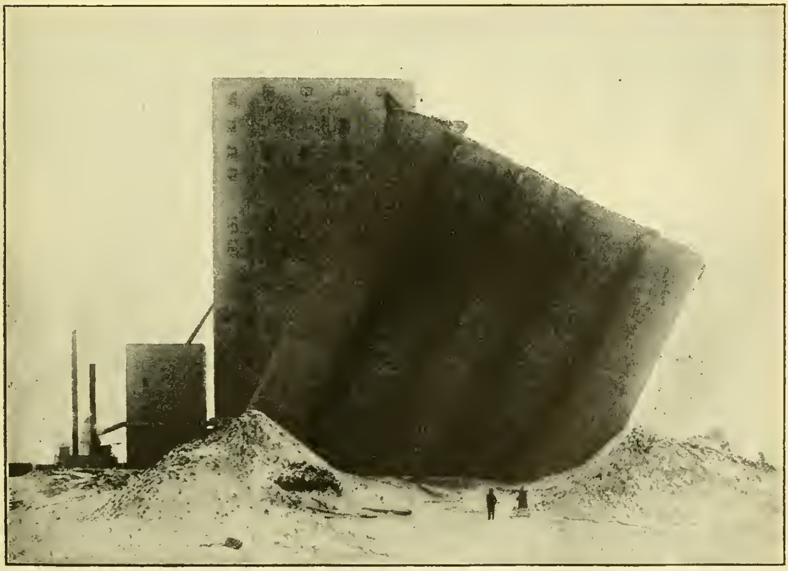
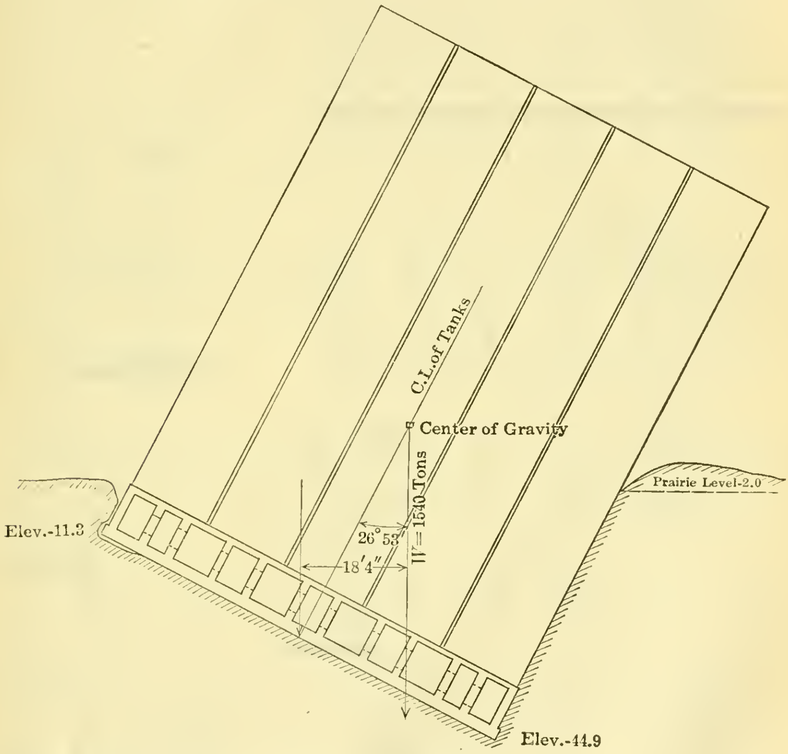


FIG. 4.—VIEW OF ELEVATOR AFTER SETTLEMENT.

In December, 1913, The Foundation Company, Limited, of Montreal and Vancouver, submitted a plan to the engineers of the railroad for underpinning the workhouse to rock, as it was feared it might fall. The plan was approved, and work was started immediately. In general, the plan followed was the sinking of a pier under each column of the building. Because of the heavy loads, the height of the structure, and its small base, it was necessary to shore the building before starting the underpinning operations at the twenty-four columns.



SECTION THROUGH BINS AFTER SETTLEMENT

FIG. 5.

The shoring consisted of pushers from twenty small piers placed outside the building columns, as shown by Fig. 6. On account of the uncertainty of the condition of the material through which the piers would have to be carried, it was decided to eliminate any risk by sinking the first six with cast-iron shoring cylinders. The cylinders

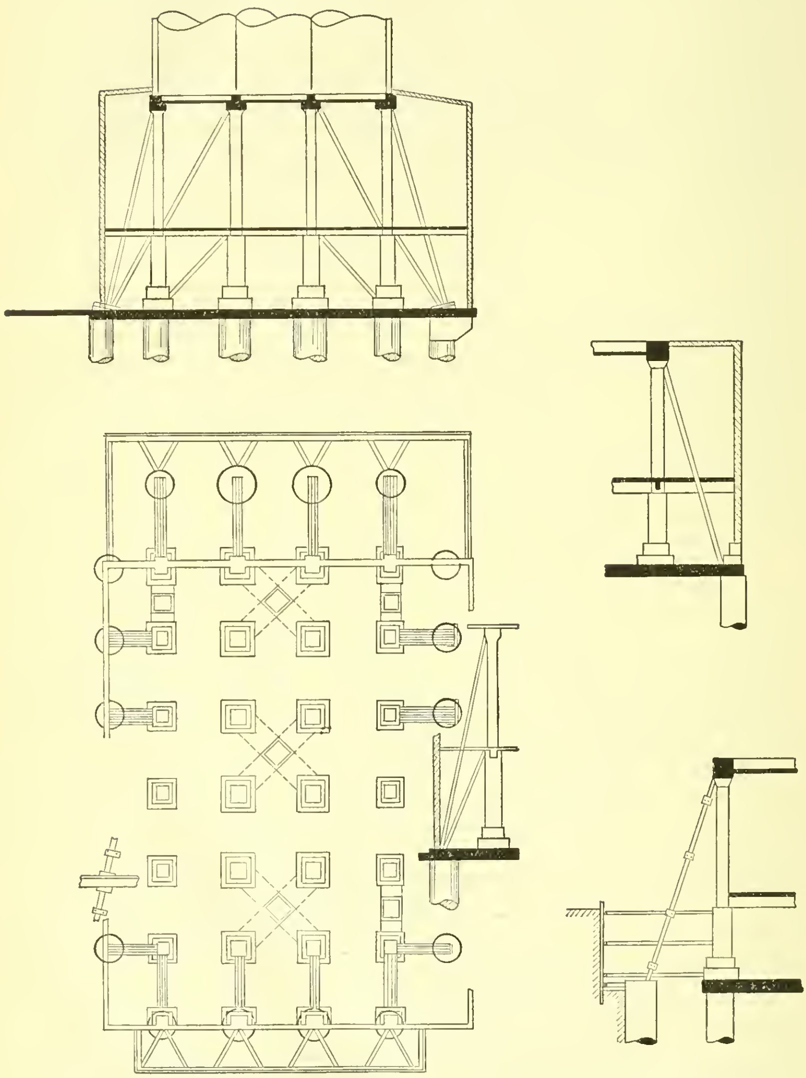


Fig. 6.

were 4 ft. in diameter, and built up in 3-ft. sections. Flanges were cast at each end of the sections for making connections. The one exception to this was at the bottom section, on which one flange was omitted, the unflanged rim being used as a cutting edge.

The shoring piers on the south and east sides were placed under the shed walls. Work on these was started by cutting a hole 4 ft. 6 in. square through the concrete floor, then drifting to the site of the pier. This excavation was sheeted, and the first section of the cylinders was placed in position. As the earth was excavated from within the shoring cylinders, the latter were jacked down by 100-ton jacks. When the top of each cast-iron section reached the ground level, another section was bolted on, and the operation was repeated. As a result of sinking six wells at different points throughout the area of the building it was decided that the Chicago well method, using wells 5 ft. in diameter, could be adopted.

The wells, with the exception of those on the south and east sides of the building, were placed clear of the walls. On the north side the work was carried on by cutting through the mat and sinking the wells immediately under it. At the west side of the building a sheeted pit was dug, approximately to the mat level, and the wells were started from this elevation.

Considerable water was encountered in sinking a majority of the shoring and underpinning wells. This was handled by No. 3 Pulsometer pumps, in some instances as many as three being required in a single well.

On each completed shoring cylinder there was placed a heavy timber shore. These were built in most cases of six 12 by 12-in. timbers, 40 ft. long, tied together with bolts and plates. Each pusher was heeled on oak blocking, placed on the top of the shoring pier at the basement floor level. Passing through an opening cut in the floor of the workhouse at the prairie level, it engaged an oak header, reinforced on two sides with 12-in. channels, the header being let into the column just below the bin floor by notching out the mushroom top, as shown by Fig. 7. Supplementing this was another pusher, extending from the shoring pier to the wall column at the first floor, and from this floor to the top of the interior column at the second-floor level.

Similar shores were placed on the shoring piers on all four sides of the building. These, combined with the columns, bin floor, and

reinforced mat, formed a truss arrangement by which the load of the building was largely transferred to the shoring piers. Later, while the interior piers were being sunk, timbers (shown by the dotted lines) were put in to take the load from the interior column bases.

All the shoring having been placed, the work of sinking the 5-ft. wells under the wall columns and the 6 ft. 6-in. ones under the interior columns was commenced. The columns are 15 ft. from center to center. To gain access to the under side of the exterior columns, holes 3 ft. 6 in. square were cut through the concrete floor mat between pairs of columns. Drifts 4 ft. wide and 6 ft. deep were driven in the direction of each column. As a well site was reached, the space was enlarged to a circular area slightly greater than the outside diameter of the well lagging. These drifts or tunnels were sheeted to prevent loss of ground, and afforded the only means of communication with the wells. The excavated material was hoisted in light galvanized-metal buckets which traveled on curved tracks or skidways, necessary on account of the center of the top of the underpinning well and opening in the floor being off center by 7½ ft. At the basement floor level the spoil was dumped into wheelbarrows, which were wheeled to a motor hoist, raised to the prairie level, and wasted at a short distance.

Prior to opening up the wells at the workhouse, levels had been taken on the column footings. As the piers were sunk, check levels were taken at regular intervals. As a further check on any movement, a plumb-bob weighing 280 lb. was suspended from the roof of the building down through an elevator shaft. The total distance was about 160 ft. The plumb-bob was immersed in a tank of water which was prevented from freezing by an electric coil. The position of the bob was noted every day.

Wells under the interior columns were approached in the same manner, except that the holes through the floor mat were cut centrally with respect to four columns, and handled the wells under them by drifting to each site in turn as one after another of the wells was completed. The distance from the center of the floor opening to the center of each underpinning pier in this instance was 10 ft. 6 in. Special curved skidways were used for these wells also. In all cases single-drum hoists, electrically operated, and having a rope speed of 130 ft. per min., were used to elevate the excavated material.

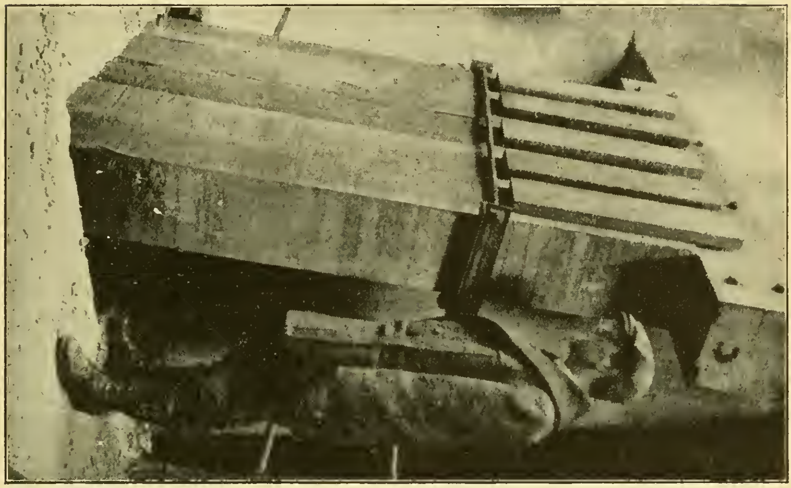


FIG. 7.—TYPICAL TOP OF WORKHOUSE PUSHER.

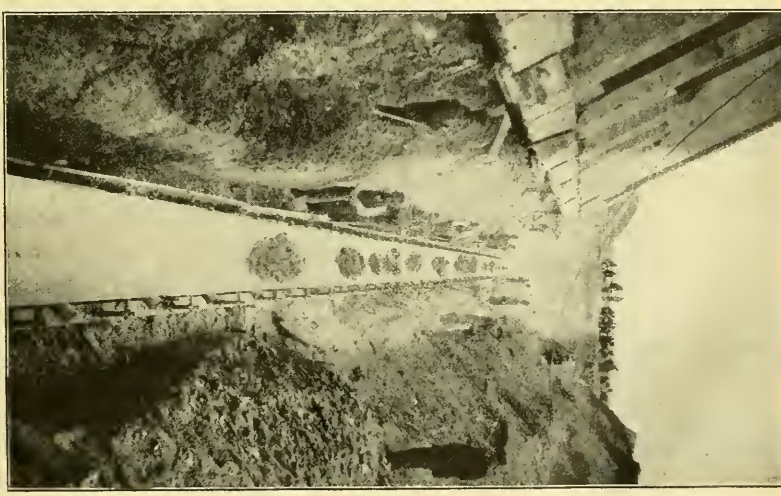


FIG. 8.—TRENCH, WITH BELT CONVEYOR, ALONG EAST SIDE OF BINHOUSE.

Concrete, up to the top of the wells, was deposited through light 8-in. galvanized-iron pipe, cut to short lengths to accommodate them to the necessary curvature. Above this elevation, and up to within 8 in. of the under side of the columns, concrete was shoveled into place behind forms, the near side of which was built up as the concreting progressed. After this concrete had set sufficiently, the remaining 8 in. were finished both by grouting and by ramming in fairly dry concrete.

The workhouse operations were completed by about the beginning of June, 1914.

In the latter part of Feburary, 1914, permission was given to The Foundation Company, Limited, to proceed with the straightening of the binhouse. When the vertical position had been reached, it was planned to underpin it also by concrete piers to rock, these piers to be placed under the contact points of the tank walls in longitudinal rows. As a matter of economy, it was decided not to attempt to raise the building to its former elevation, but to straighten the structure by rotating it about the low edge; this was to be accomplished by excavating under the east or high side, and lowering it to the level of the low or west side. This meant that the mat in its final position would be approximately 38 ft. below the prairie level. As this was below the ground-water line, it was proposed to water-proof it.

The binhouse when empty of grain weighed 20 000 tons. Under the lower or west edge of the mat fourteen piers were sunk to rock and concreted. It was the intention to block the building off these piers to form a fulcrum, about which it would rotate as the excavating was done under the high side. The clay actually under the mat was removed in two ways: First, by working from under the high edge of the mat; and secondly, through holes in the mat. The former necessitated the excavation of a trench paralleling and flush with the east edge, as shown by Fig. 8. This was carried to a depth of about 8 ft. below the mat edge, was approximately 10 ft. wide at this point, and sloped back on the side away from the structure to the natural angle of repose of the soil. Drifts were driven from the trench toward the west under the mat. In the trench was placed a belt conveyor which discharged upon another belt, at the north end of the building, running at right angles to the first one. This second belt emptied into a hopper from which the earth was hauled by team

and spread over the prairie. The holes in the floor were in the compartments formed by the cross-walls between those of the main tunnels. The spoil passed through the holes and was handled by two conveyor belts, one in the first and the other in the third tunnel from the east. These belts discharged upon the transverse belt, previously mentioned as running along the north end. The gradual settlement of

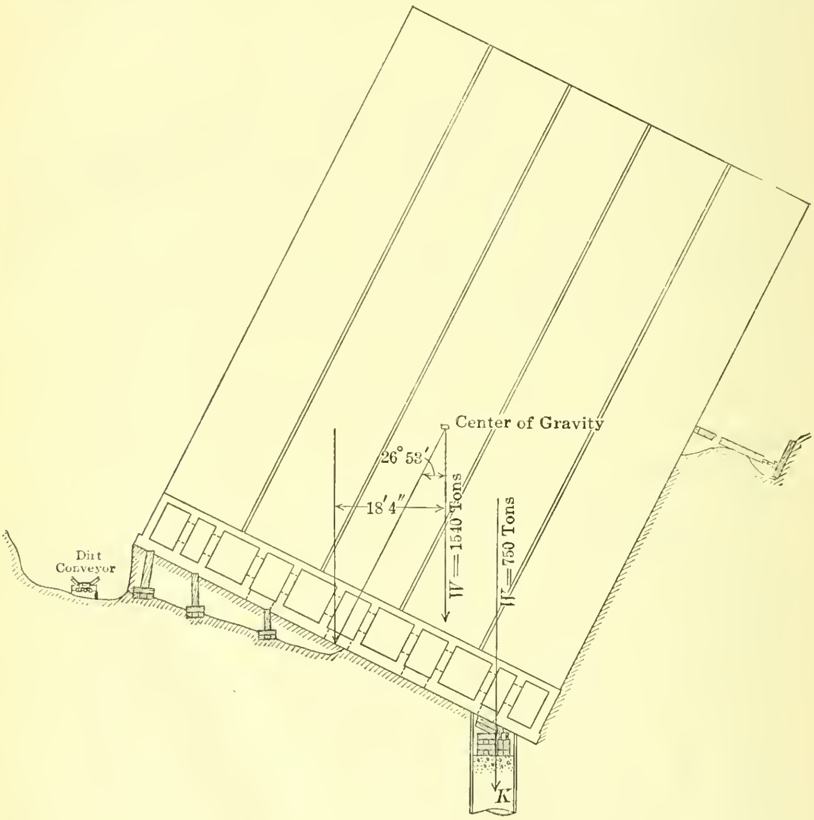


FIG. 9.

the high side was to be accomplished by weakening the core of earth left between the drifts. In driving the drifts, the earth wall between them was narrowed from the bottom up, the intention being that, as the building dropped, this core would offer increasing resistance to the movement, and that this movement could be controlled by removing the earth from the sides of the cores.

The fourteen piers under the low side, known as the *K* row, were placed on the longitudinal center line of the thirteen tanks, as shown by Fig. 9. They were placed under the contact walls and at the ends. Access to the sites of these wells was secured by cutting holes, 3 ft. 6 in. by 4 ft., through the mat in the compartments between the tunnels proper. The experience gained in sinking the workhouse wells showed that a large quantity of water would have to be handled in sinking the wells of the *K* row. Preparations for doing this were made by lagging down a large sump with 10 by 10-in. timbers at the north end of the building. This sump was carried to the depth of the low corner of the mat. It was placed inside the excavation which had been made to put in the grain leg and also the transverse belt conveyor used in connection with handling the spoil. In the sump was erected a duplicate set of electrically-driven centrifugal pumps and steam-driven piston pumps. Each set had a capacity of 1 200 gal. per min.

In sinking the wells of the *K* row, as well as the others described later, the work was performed under considerable difficulty, owing to the 27° inclination of the mat and the confined working space. Spoil from the different wells was raised by hand windlasses. The limited space and the small quantity of material to be handled would have made any mechanical arrangement more expensive.

The depth of sinking at the *K* wells was only 14 ft. Lagged wells of the Chicago type, 7 ft. in diameter, were used. The material was found to be very firmly compressed; in some instances the blue clay had been driven into the white stratum. The white clay, instead of being soft, as it had been found at the workhouse, was squeezed dry and hard, so that it was usually necessary to pick before shoveling into the buckets. No water was found until the shattered limestone stratum, immediately overlying the rock, was reached.

Not only was a large quantity of water found in the *K* wells, which from No. 1 to No. 6 came in at the rate of 1 150 gal. per min. in each well, but, in addition there was the danger that the level to which it would normally rise was approximately 14 ft. above the mat at the openings. This made imperative the duplicate pumping system at the sump. The insufficiency of room further aggravated the water troubles.

To do away with the multiplicity of piping of the small steam units and the almost unbearable heat from them, belt-driven, electrically-operated centrifugal pumps were erected immediately above the wells. These were placed on the main tunnel floors, and independent holes for the pump suctions were cut through the mat. As it was considered inadvisable to open up too many wells at one time, alternate ones only were started. As the concreting of these piers was finished, oak cribs were placed upon them to take the load of the building, as shown by Fig. 9.

As the work on the *K* piers was nearing completion and the majority of them had been blocked up, the construction of the remaining 56 piers was commenced, it being the intention to sink these piers while the righting process was under way. These were opened up as far as possible in widely scattered positions. Although quantities of water were encountered, particularly at the north end of the building, the working conditions were somewhat better than at the *K* row, and the work progressed steadily. These wells were also 7 ft. in diameter; they were placed under the contact walls and tank ends, and were in all cases carried to rock. They were arranged in four rows, starting from the east, and designated as *G, H, I,* and *J.*

At about this time the original plans were changed, owing to the fact that the railroad engineers had decided that they would prefer to have the tunnel floors above the ground-water level. To accomplish this the new plan contemplated allowing the bins to rotate about the *K* row until the angle, 16° 30′, had been reached, at which time the binhouse was to be pivoted about the *I* row of piers to an angle of 8° 30′. At this point the structure was to be again pivoted about the *H* row of piers into a vertical position.

As the drifts or tunnels were driven under the high side of the mat, 12 by 12-in. posts were set up in them on mudsills of such an area that the load imposed on the soil was about 3 tons per sq. ft. See Fig. 10. These helped to make up for that resistance against the mat which was lost as the drifts advanced. Later, as the excavation was extended so that the sinking of the wells in the *G, H, I,* and *J* rows could be proceeded with, more posts on mudsills were added, for the same reason.

The number of posts put in also enabled the settlement of the structure to be closely regulated. This was preferable to depending

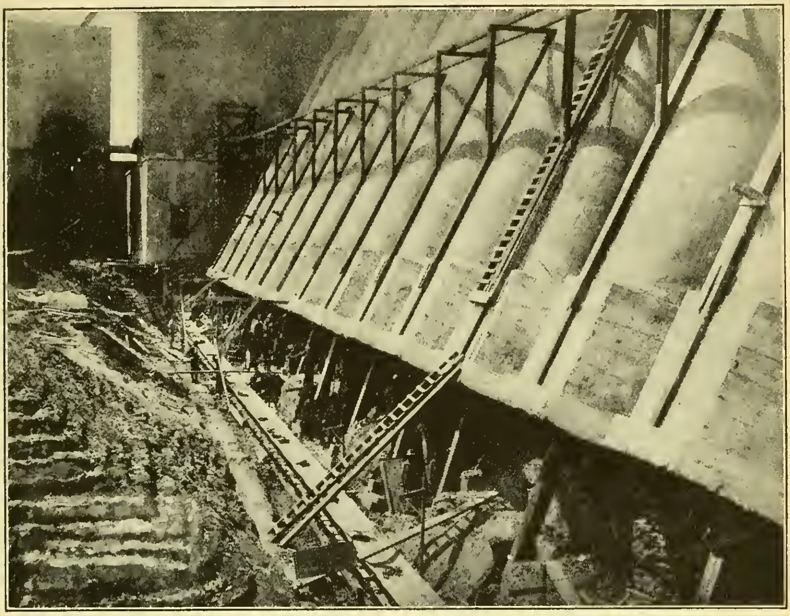


Fig. 10.—Excavation Toward West from Under East Edge of Mat.

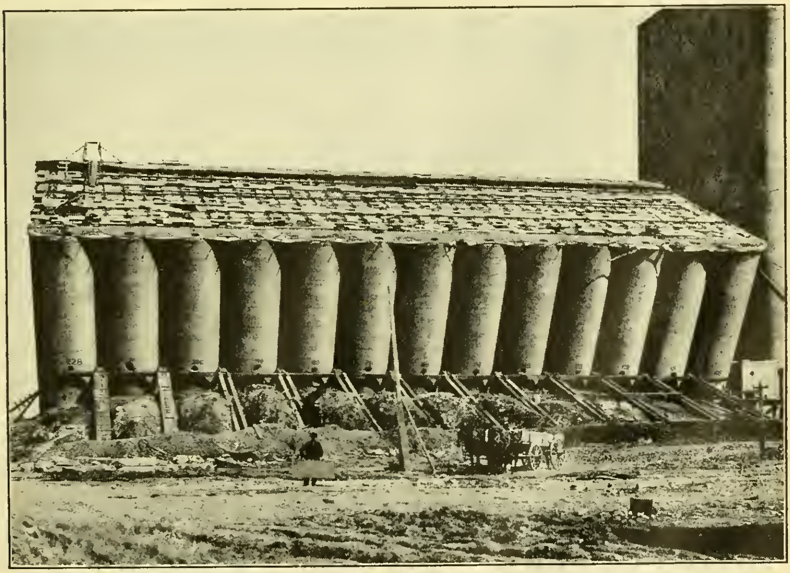


Fig. 11.—Line of Pushers Against West Side of Bin Structure, Showing Holes through which Grain was Taken Out.

only on the clay ribs, which, on occasions, had a tendency to break off in large masses. This fracturing of the clay cores also necessitated increasing the number of posts.

The posts as well as the shoring screws, mentioned later as being placed on the *G* and *H* piers, were also necessary on account of the fact that the reinforced mat, tunnel walls, and bins were not tied together vertically in any manner. Without some such precaution the mat could have broken off, or the three parts could have slid on one another.

With the bins at rest at the angle of 26° 53′, calculations showed that the loads at the pier sites were as follows:

Those in *G* Row	......	Uplift.
" " *H* "	......	11 tons per pier.
" " *I* "	......	227 " " "
" " *J* "	......	529 " " "
" " *K* "	......	773 " " "
Total per transverse row	....	1 540 tons.

To assist in righting the bins, twelve pushers, as shown by Fig. 11, were placed against the west side of the tanks. Each of these engaged a 12 by 12-in. waling piece placed against the side of the tanks about 45 ft. down from the top, this distance being selected so that their forces would be in a perpendicular direction to the vertical height of the tanks. The wale, in addition to resting against the thirteen tanks, was also posted into the contact walls, for it was at these points that the pushers were applied. Each pusher was composed of two 12 by 12-in. timbers, spaced 12 in. apart by using spreaders of that dimension, and tied together by plates and bolts. Each was 60 ft. long, and each had two screws, one in each of the 12 by 12-in. timbers, these screws heeling against ample timber mats. The screws were operated until the mat had reached an angle of 8° 30′. The line of action of the pushers was lowered once during the righting operation so as to maintain as nearly as possible their perpendicular direction against the sides of the tanks.

The initial righting movement, namely, that of rotating about the *K* row, was induced by weakening the earth partition between the drifts. Practically, a continuous daily movement of from 3 to 4 in.,

at the eastern edge, was maintained. When the bin floor had reached an angle of 18°, the movement was stopped for the purpose of placing the hardwood rockers at the *I* row of piers.

During this initial movement all the wells under the mat were successfully bottomed and concreted to the heights required for carrying on the work. As rapidly as the *G* and *H* rows were completed, cribwork was placed on each pier, with 40-ton shoring screws in contact with the mat, as shown by Fig. 12. These screws made possible a much more positive control in lowering the mat than could have been attained by the clay rib alone. The placing of the shoring screws and posts on mudsills also permitted the removal of all the clay under the mat back to the west side of the *H* row of wells, approximately 23 ft. from the edge of the mat. The removal of this excavation from under the eastern edge of the mat was thus carried on more economically than if the excavation had been made through the holes in the mat. In fact, as the lowering of the mat continued, a uniform depth of the clay of approximately 8 ft. from the outer edge and 4 ft. at the line back of the *H* row was maintained—the belt conveyor used for the disposal of the material being also lowered from time to time as the general excavation progressed.

Shortly after starting to rotate the structure about the *K* row, readings showed that a lateral or sliding movement to the eastward had set up. This was hard to understand, on account of the inclination of the bins to the west; also because of the compressed clay fronting the under side of the mat, and the fact that the dead load of the structure was 20 000 tons. Notwithstanding this, however, the tendency was always in evidence until the structure was pivoted on the *I* row of piers. To resist the movement, 12 by 12-in. timber kicking braces (Fig. 13), usually arranged in pairs, were used. These were of two lengths: short ones engaging the high edge of the mat, and long ones reaching in under the mat to notches cut in its under side. as shown by Fig. 14. In both cases they heeled against the earth embankment on the far side of the trench, where liberal timber heels had been set up. To retard the lateral movement still more, short inclined posts were set up on the *K* piers, their upper ends engaging notches cut in the mat. In addition, approximately 3 000 cu. yd. of earth, pressing against the bins along the west side, were removed. This total lateral movement to the west finally amounted to 3 ft. 3 in.

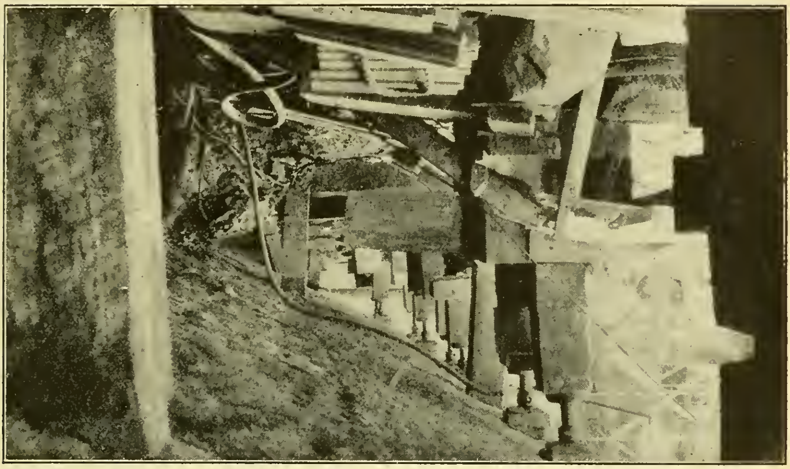


FIG. 12.—SHORING SCREWS, ON "*G*" ROW OF PIERS, CARRYING EDGE OF MAT.

FIG. 13.—KICKING BRACES AGAINST EASTERN EDGE OF MAT AT BINHOUSE.

On August 13th the structure was brought to rest at an angle of 18° by using cribbing placed on the *J* row of piers. There were two of these cribs on each pier, and they were placed in a direction transverse to the longitudinal axis of the bins. They had a total area in plan of 4 by 7 ft. The rear one was 2 by 4 ft., and was removed later to give room for the shoring screws; the former one was used for following up with blocking as the structure rotated about the *I* and *H* rows of piers.

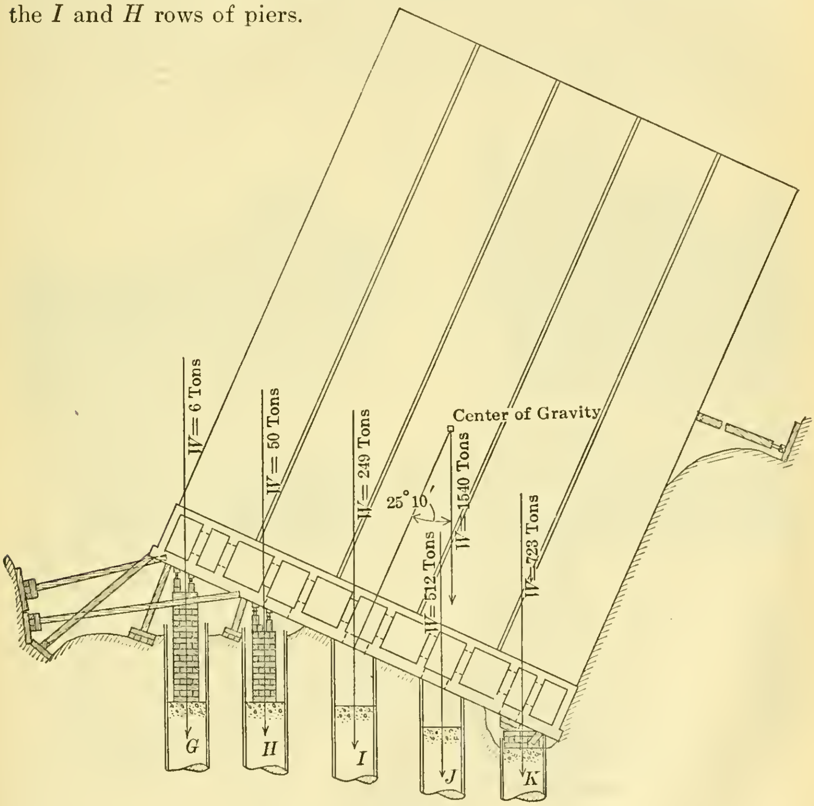
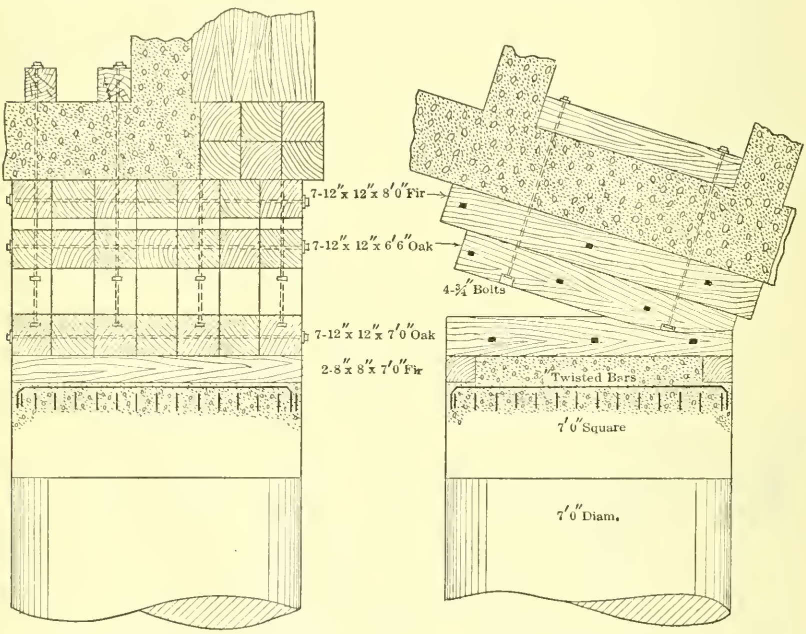


Fig. 14.

To form convenient working places, all the tops of the piers were made 7 ft. square. In preparing the piers for the rockers, as shown by Fig. 15, 8 by 8-in. timbers were embedded in the edges parallel to the longitudinal axis of the bins. The rockers consisted of two parts: the shoe and the rocker proper. The former was made by covering the top of each pier with 12 by 12-in. oak timbers, laid in

the direction of the movement and strongly bolted together. The upper surface of this shoe was concaved to a radius of 30 ft., and also in the direction of the rotation.

The rockers consisted of two courses of 12 by 12-in. oak timber, seven pieces in each course, directly above the timbers in the shoe. These two courses were strongly bolted together and also to the mat. To make certain of an even bearing against the rough bottom of the mat, all uneven places were grouted. The lowest position, or heel, of the rocker had previously been rounded to a radius of 15 ft. Thus the area of contact of the two surfaces was 2 ft. 6 in. by 7 ft. 0 in.



ARRANGEMENT OF TIMBERING ON "I" PIERS

FIG. 15.

Clearance was allowed between the rockers and the shoe, so that contact was not made until the bins reached an angle of 16° 30′. This allowance was made in order to provide for the possibility of a slight settlement during the time required to set up the rockers. When the mat had reached this angle, the center of gravity had shifted to a position 11 ft. 3 in. west of the center of the structure. At the angle

of 26° 53′, it had been 18 ft. 4 in. to the west. In the new position the figured pressures on the piers (see Fig. 16) were as follows:

G......	80 tons.
H......	193 "
I......	316 "
J......	423 "
K......	528 "

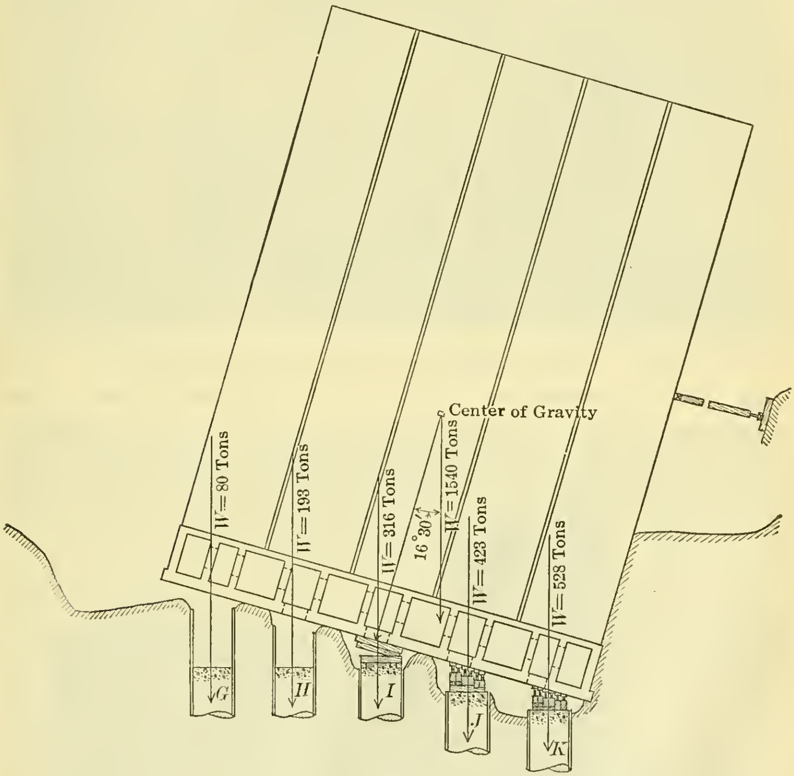


FIG. 16.

The rockers on the *I* row having been completed, the next step consisted in preparing the *K* and *J* rows for the shoring screws. Excavation to a depth to give ample room was made from the east face of the *J* piers to the west face of the *K* piers. This excavation stepped up as the *J* piers were approached, so as to maintain a practically constant head room and reduce the quantity of back-fill required later.

Between the piers of the *K* row the excavation exposed the white clay, previously mentioned as having been very much compacted. Over the clay and extending from pier to pier throughout the total length of the *K* row was laid 1 ft. of concrete. On these slabs were placed oak timbers, and on the latter four 50-ton shoring screws were set up. Twelve 50-ton screws, surrounding oak cribs 4 ft. square, were placed on the *K* and *J* piers. At the time of starting the movement there were, therefore, twenty-eight jacking units of twelve screws each and thirteen of four screws each, as shown by Fig. 17.

The 4-ft. square oak cribs were used to block up the mat as fast as the movement took place.

At the time of landing on the *I* row of piers, the mat to the east of this row had been brought in contact with the clay. From this time forward the overhang to the east of the center row was carried entirely on the clay, the shoring screws from the *G* and *H* piers having been removed to assist in jacking up at the west side. Prior to any movement of the bins, and while they were being righted, calculations were made to ascertain the weight of the structure at the different piers. These calculations were made for every 30′ of arc. From the calculations and a knowledge of the clay values, it was possible to maintain such resistances to the advance of the building as would just support the mat and prevent it from fracturing as a result of overhanging the *I* rockers. Before pivoting on the *I* row of piers, these studies also had governed the number and operation of the shoring screws and posts.

Further, the calculations of the weights at the different angles were applied to the conditions which would exist at the *I* and *H* rows of piers when used as fulcrums. These showed that the loading approached too closely to the safe value of the oak. The loading at the rocker points, therefore, was lessened by maintaining a calculated resistance of the clay under the east side. Extra screws under the low side were used to overcome this resistance.

The righting operations were continued, both by lessening the resistance of the clay under the overhanging portion of the mat and by jacking up the low side. Trenches were excavated under the mat from the *H* rows of piers eastward. These trenches extended to within about 3½ ft. of the east edge of the mat, and averaged 4 ft. in depth, except when the mat was approaching the horizontal position, when

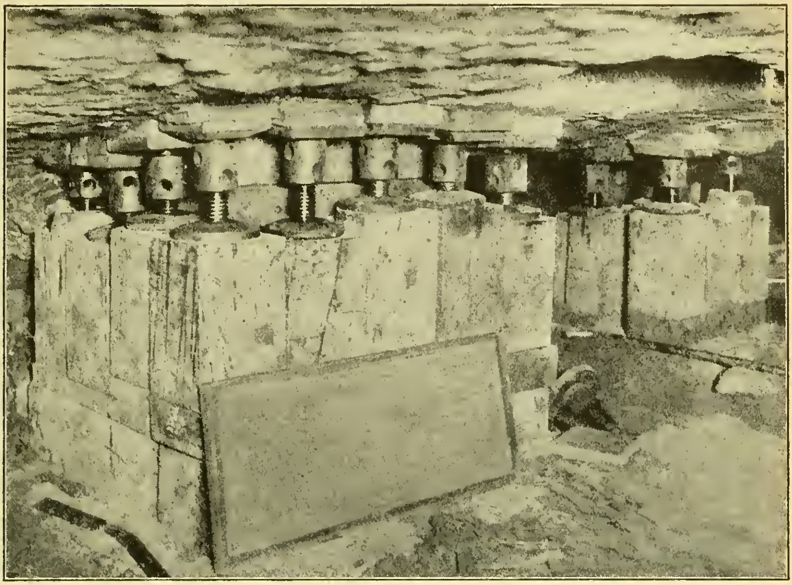


FIG. 17.—TYPICAL JACKING UNITS ON AND BETWEEN "*K*" PIERS.

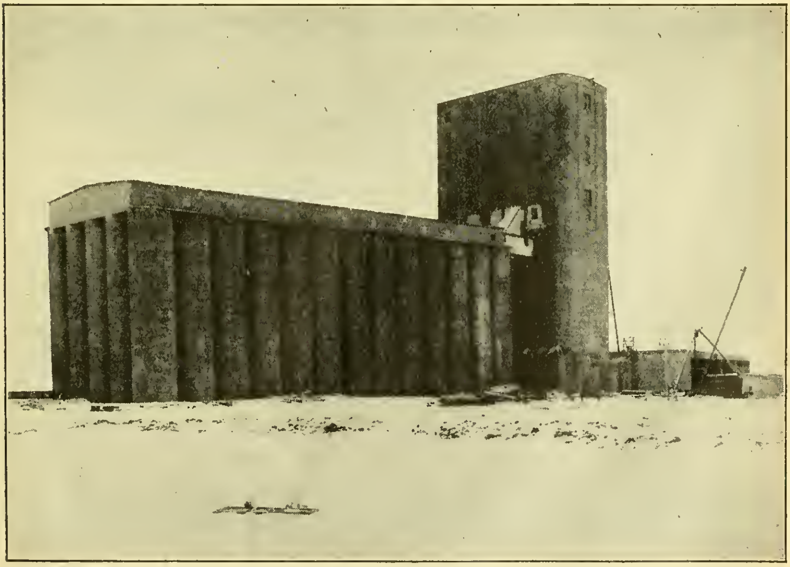


FIG. 18.—STORAGE BINS IN RIGHTED POSITION.

they were allowed to fill. The bank of clay between the *H* and *I* rows was left in place. The width at the bottom of the trenches averaged about 3 ft., the sides being sloped so that each rib between two trenches had a triangular section. Continuous weakening of the clay ribs and deepening of the trenches was maintained at a rate practically equivalent to the uplifting thrust of the shoring screws. Spoil from the trenches was handled through holes in the mat.

The jacking part of the lifting operation was done by gangs made up of three men. Each gang was supplied with a 6-ft. steel bar $1\frac{1}{8}$ in. in diameter, which, when inserted in the head of the shoring screw with three men pulling on it, was equivalent to an effort of approximately 55 tons. In the *K* row each gang handled eight screws, and six was the allotment for the *J* row. The periods of work and rest were carefully arranged, and a uniform application on the screws throughout the total length of the bins was maintained during such periods. The jacking-up process was only carried on during the day shifts. At the same time, six gangs of two men each were kept busy fleeting the screws and following up with oak blockings as progress was made. The night shift was utilized to fleet the screws, block up, and add concrete on the tops of two piers each night, and on both the *J* and *K* rows. The clay filling around the piers, to maintain the earth floor to the proper working height, was also done during the night shift.

The position of the structure was checked twice daily. In addition to levels being taken at numerous points around the structure, two verniers, fabricated on the work, were attached to the east walls inside of the north and south tanks of the *G* row. Each vernier consisted of an arc of a circle of 15 ft. radius, graduated to 5′ of arc. A wire with weighted lower end hung from the center of the circle. Prior to any movement of the bins, this had been set to 26° 53′. Thus the actual angle of the structure could be ascertained quickly at all times. A 240-lb. plumb-bob was also suspended from the top edge of the west face of the bins. By these devices—in addition to the angle changes—any warping of the structure, failure of the tanks to follow the mat movement, and bending of the mat could be noted.

On September 17th the angle had been reduced to 8° 30′. Jacking was discontinued while oak rockers, similar to those used on the *I* piers were placed on the *H* piers. As in the previous case, at the *I*

piers, allowance had been made for the creeping of the structure during the time it took to place the rockers, and they did not come to a full bearing until the angle 7° 30′ had been reached. The shoring screws were now removed from the pushers at the west side of the building and set up on the *I* piers. The grouping of screws during this, the final movement, was as follows: *K* piers, each twelve screws; *J* piers, each ten screws; and *I* piers, each eight screws. At this angle (see

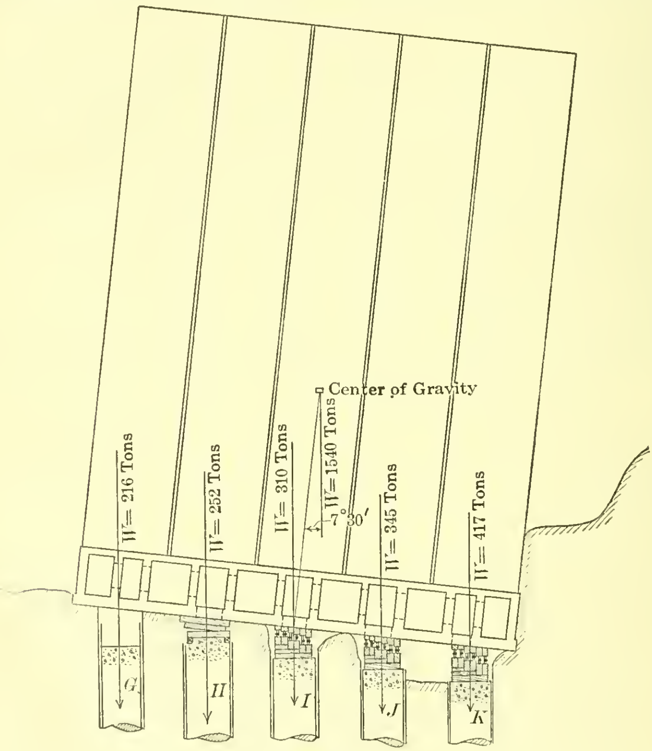


Fig. 19.

Fig. 19) the pier loading was as follows: *G*, 216 tons; *H*, 252 tons; *I*, 310 tons; *J*, 345 tons; and *K*, 417 tons. The center of gravity, in the meantime, had shifted until at this angle it was only 3 ft. 3 in. west of the center line of the structure. During this last operation it was found advisable to assist the screws on the *I* piers by using oak wedges, 4 ft. long, 6 in. wide, and 3 in. thick at the butt end; these were driven between the shoe and the rocker on these piers. This was made necessary on account of the tendency of the mat to sag.

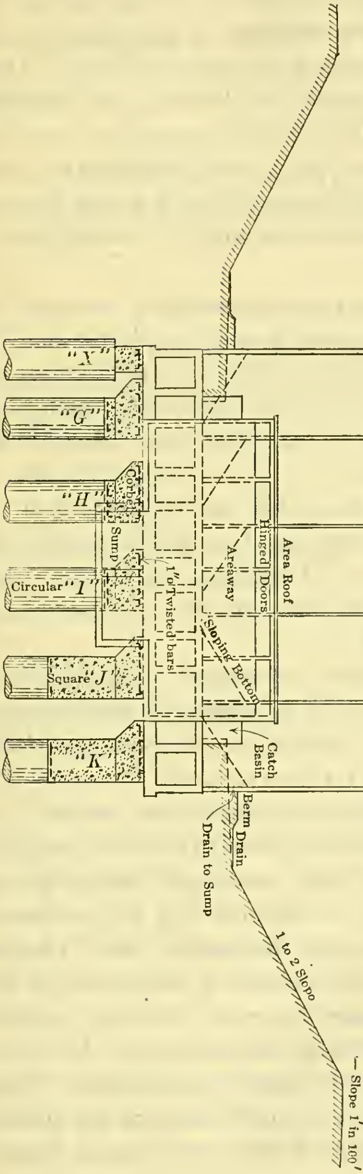

"X"
"G"
"H"
Corbel
Sump
Circular "I"
Square "J"
"K"
1" o' Twisted bars
Areaway
Hinged Doors
Area Roof
Sloping Bottom
Catch Basin
Berm Drain
Drain to Sump
1 to 2 Slope
Slope 1' in 100'

FIG. 20.

An average rate of 4 in. in 10 hours, measured on the center line of the K piers, was maintained throughout the jacking portion of the righting operation. The binhouse was back in its proper vertical position on October 17th, 2 days behind the estimated time. After the building had been righted, the screws and blocking were removed from piers in scattered locations. In concreting up to the under side of the mat, these piers were corbeled out, so as to distribute the pressure, as shown by Fig. 20. The building as it stands to-day is practically 14 ft. below its original position, the total lift having been a trifle more than 12 ft.

To meet the new elevation of the tunnels, excavation was made in the workhouse, and the grain boots were lowered to receive the discharge from the tunnel belts. Also, to provide for the overhang of the bins on the east side, caused by the lateral movement, seven piers were constructed with corbeled tops to carry the outer points of the tanks.

The inclination of the bins to the north, amounting to 4 ft. in their total length, was allowed to remain. This does not affect the stability of the structure, nor the cost of handling the grain, so that any additional expense to get the structure exactly level north and south would have been unwarranted.

At the north end of the structure, an areaway, 6 ft. wide and 52 ft. long, was built to a height of 15 ft. above the tank bottoms. This was roofed over with concrete, 2 ft. 6 in. above the top of the walls, the open space being left open for the ventilation of the tunnels. At the bottom of the areaway was built a sump. This extends 7 ft. below the mat, and is 6 ft. wide and 21 ft. long. Here were installed two No. 4, motor-driven, submerged type, centrifugal pumps, having a rated capacity of 470 gal. per min. As shown by Fig. 20, no attempt was made to back-fill around the tanks to the prairie level. Leaving the depression as shown precluded the necessity of water-proofing the tanks, which would have been necessary to protect the grain had the soil been allowed to come in contact with the thin walls. It will be noted that a berm 8 ft. wide was left around the structure. This slopes away from the bins, and forms a drain for carrying the surface water to catch-basins placed just outside of the north areaway, whence it is piped to the sump.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

A REVIEW OF THE REPORT OF CAPTAIN ANDREW TALCOTT CHIEF ENGINEER MEXICO AND PACIFIC RAILROAD EASTERN DIVISION FROM VERA CRUZ TO MEXICO EXPLORATIONS SURVEYS ESTIMATES 1858*

By Emile Low, M. Am. Soc. C. E.

Synopsis.

The object of this paper is to record the achievements of an American engineer in railroad building in a foreign country under crucial conditions, to render him proper recognition, and to pay the tribute due him.

The first railroad built in Mexico was what is now called the Mexican Railway, extending from Vera Cruz to the City of Mexico, a distance of 264 miles. Barring the two short stretches at each terminus, built prior to 1858, this railroad was constructed during the period 1861-72. The construction was intermittent, having been interrupted at times by civil strife and again on account of financial difficulties.

Under the auspices of the President of the Mexican Republic, imposing ceremonies commemorating the completion of this railway were held in the City of Mexico, on January 1st, 1873, on which date it was officially opened for traffic. Between Orizaba and Boca del Monte

* This paper will not be presented at any meeting of the Society, but written communications on the subject are invited for subsequent publication in *Proceedings*, and with the paper in *Transactions*.

occurs the famous Maltrata Incline, which, as an engineering achievement, is without parallel in railroad construction. This section, together with other portions of the railroad, is an everlasting monument to the ability of Capt. Andrew Talcott, an eminent American civil engineer, who located the railroad and also constructed the major portion of the line.

Before giving a short biography of Capt. Talcott, attention is invited to extracts from the books of two well-known authors who visited Mexico and whose descriptions of this wonderful section of railroad find a responsive chord in the breast of every one who has passed over it.

"A little valley lies spread before us now, an emerald embosomed in the mountains, called *La Joya,* the Jewel, in the center of which is the station of Maltrata.

"Beyond this the track literally *climbs* the mountain, approaching it by great curves. All the way up the hills you can trace the road, its serpentine trail drawn in and out the valley and along the ridges, ever and anon doubling upon itself, but ever climbing. Beyond a water tank is a narrow bridge, ninety feet long, and spanning a chasm that ends only at the valley below, two thousand feet downward.

"Glorious are the views of Maltrata obtained as the train crawls in and out of the cuts. In the last thirteen miles we have climbed over three thousand perpendicular feet."*

"But our way lies onward towards the mountains. A wildness of landscape, unpictured before, opens to the view. Here rise weird rock forms, Nature's cathedral towers and grim façades, magnificent in solitude, and awe inspiring.

"Still onward and upward lies the way. One of the most remarkable railways in the world ascends to gain this steep zone, and serpentines among sheer descents to gain the summits of abrupt escarpments, from which (a remarkable feature of the topography of the eastern slope of Mexico) the traveler looks down as into another country and climate, upon those tropical valleys which he has left below. This is the Mexican Vera Cruz railway."†

Capt. Talcott was born in Connecticut, on April 20th, 1797. He entered the West Point Military Academy on April 9th, 1815, and was graduated and promoted in the United States Army to Brevet Second Lieutenant, Corps of Engineers, on July 24th, 1818, and to Captain, on December 22d, 1830. He resigned his commission in

* "Travels in Mexico", by F. A. Ober, 1884.

† "Mexico", by C. Reginald Enock, 1909.

the Army on September 21st, 1836. During this time, he had had charge of many engineering projects, and supervised the construction of important fortifications, among which were the defences at Hampton Roads, Va., Fort Adams, R. I., Fort Delaware, and Fort Monroe, serving on the last named as Superintending Engineer of Construction, from July 20th, 1828, to July 31st, 1834.

He was also the author of the "Talcott Method" of determining latitude, of which practical use was first made by him as Astronomer in the determination of the boundary line between the States of Ohio and Michigan, in 1832-36.

During his civil life, among many other engagements were those of Adjunct Chief Engineer, New York and Erie Railroad; Chief Engineer, Richmond and Danville Railroad, 1848-55; Chief Engineer, Eastern Division, Ohio and Mississippi Railroad, resigning from the latter to become Chief Engineer of the Mexico and Pacific Railroad, on December 1st, 1857, at a time of life when most men think seriously of retiring from active duties. After the presentation of his report in April, 1859, Capt. Talcott was otherwise engaged, but resumed his connection with this railroad early in 1862, resigning his position on February 10th, 1867, when the nearly completed work was suspended at the time of the withdrawal of the French Army from Mexico.

He died on April 22d, 1883, in Richmond, Va., being at the time the oldest graduate of the West Point Military Academy.

The paper is divided into eight parts, as follows, and, for a clearer understanding, the main reports are given verbatim, in order that there may be no perversion of facts:

1.—Introduction.
2.—Historical.
3.—Physiography.
4.—Explorations and Surveys.
5.—Report of Capt. Andrew Talcott.
6.—Summaries of Special Reports, *A, B, C, D, E,* and *F.*
7.—Maps, Profiles, Plans, and Estimates.
8.—Appendices *A, B, C, D, E,* and *F.*

It is opportune to state here that the original records used in the compilation of this review are now the property of Col. T. M. R. Talcott, of Richmond, Va., one of the two living sons of Capt. Talcott,

and a former member of the American Society of Civil Engineers, who kindly furnished the data for publication. Other data were obtained from various sources.

1.—Introduction.

In all civilized countries, before the advent of railroads, highways of more or less elaborate construction formed the means of communication. Mexico was no exception to this rule. Two highways, by divergent routes, connected Vera Cruz, the seaport of Mexico, with the City of Mexico, the capital of the country. One of these highways, known as the northern, passed to the north of the mountain, Cofre de Perote, (12 500 ft. high), and the other, the southern, to the south of the mountain, Pico de Orizaba (17 873 ft. high). A straight line drawn from Vera Cruz to the City of Mexico passes about midway between these extinct volcanoes, the air-line distance being about 200 miles.

The peaks of the volcanoes are about 30 miles apart and are connected by a ridge, the lowest elevation of which above sea level is about 10 000 ft. Puebla was and is now the largest city between the coast and the capital. Both highways passed through this city, or it might be more properly said, they joined here, there being only one main highway between Puebla and the City of Mexico.

North of Puebla is the mountain of Malinche, an isolated peak rising 6 000 ft. above the surrounding plain. A branch of the northern highway passes to the north of this extinct volcano, through Huamantla, joining the lesser Puebla-City of Mexico *camino real* west of Apizaco. The main city on the northern highway is Jalapa and that on the southern, Orizaba. Still farther west are the well-known volcanoes, Popocatepetl and Ixtaccihuatl, the slopes of the latter extending some miles to the north. These mountains fixed absolutely the routes of travel, first the trails used by the native Indians, next the highways built by Cortez and his followers, and finally the location and construction of railroad lines.

The highways were built about 1800 by the consulates of Vera Cruz and Mexico. Later, the one through Jalapa was virtually abandoned, mainly on account of the larger number of important towns along the southern highway. Although there was no appreciable difference in length, it took 4 days to make the journey by diligence

from Vera Cruz to the City of Mexico *via* Jalapa, and only 3 days *via* Orizaba.

These roads, notwithstanding their dilapidation due to gradual neglect, were originally admirable specimens of solid construction, as were also the bridges on them, the Puente Nacional on the Jalapa Section deserving special mention.

2.—Historical.

On August 31st, 1837, under the administration of Gen. Anastacio Bustamente (who was President from 1837 to 1839), Don Francisco Arrillaga, a merchant of Vera Cruz, obtained an exclusive privilege for the construction of a railway from the City of Mexico to Vera Cruz, with a branch to Puebla. The proposed line had one great drawback, namely, that it only united Vera Cruz, Puebla, and the City of Mexico, leaving Cordoba and Orizaba isolated on one side, and Jalapa on the other.

The projector estimated the cost of the main line at $5 000 000 and that of the Puebla Branch, leaving at Apizaco, at $500 000. After some surveys had been made, the concession was forfeited, no construction work having been done.

A new concession was granted on May 31st, 1842, by President Don Antonio Lopez de Santa Ana, for the construction of a railroad from Vera Cruz to the Rio de San Juan, in the State of Vera Cruz, about 20 miles west. A tax formerly known as *de averia* (highway repairs) was re-established, and the proceeds were applied to railroad building and to repairs of the Jalapa Highway. During the 7 years of this grant, the company holding it constructed only 1 league of railroad and then abandoned the concession, the Government confiscating the works and prosecuting them until 1854, when a tramway had been built to La Caleta (a suburb of Vera Cruz) and the railroad was extended to the San Juan River.

On August 2d, 1855, Messrs. Mosso obtained a charter to construct a railroad from San Juan (near Vera Cruz) to Acapulco, virtually from the Atlantic to the Pacific. The resulting operations were the construction of a short stretch of railroad about 3½ miles long from the City of Mexico to the neighboring village of Guadalupe Hidalgo, which was opened for traffic on January 11th, 1857. This short line and the somewhat longer one out of Vera Cruz to Tejeria comprised

at this date the entire railway system of Mexico, some 20 miles, much of which was built under the direction of the late Robert B. Gorsuch, Hon. M. Am. Soc. C. E., later an Assistant under Capt. Talcott.

The Mosso concession was sold to Don Antonio Escandon, who, on August 31st, 1857, obtained from the Mexican Government the right to construct a railroad from Vera Cruz to the Pacific Ocean. Señor Escandon also bought from the Government the line between Vera Cruz and San Juan, near Tejeria. Previous to this date all attempts to connect Vera Cruz with the City of Mexico by rail were abortive, and it may be said that the realization of what ranks as a great engineering enterprise began at that time.

The dominant spirit in this great undertaking was Don Manuel Escandon, who placed his younger brother, Antonio, at the head of it, because he realized (as actually happened) that he himself would probably not live long enough to carry an enterprise of such magnitude through to completion. The death of Don Manuel Escandon occurred in the City of Mexico, on June 7th, 1862.

3.—Physiography.

The surface features of Mexico consist of an immense elevated plateau, with a chain of mountains on its eastern and western margins, which extend from the United States frontier southward to the Isthmus of Tehuantepec; a fringe of lowlands (*tierras calientes*) between the plateau and the coast on each side; a detached, roughly mountainous section in the southwest, which belongs to the Central American Plateau; and a low sandy plain covering the greater part of the Isthmus of Yucatan. The Peninsula of Lower California is traversed from north to south by a chain of barren mountains which cover the greater part of its surface. The slopes are precipitous on the east coast, but, on the west, they break down in hills and terraces to the Pacific. This range may be considered to be a southward continuation of the California Sierra Nevada.

The Great Plateau of Mexico is very largely of volcanic origin. Its superstructure consists of igneous rocks of all descriptions with which the original valleys between its marginal ranges have been filled by volcanic action. The remains of transverse and other ranges are to be seen in the isolated ridges and peaks which rise above the

level of the table-land, in some cases forming well-defined basins; otherwise, the surface is singularly uniform in character and level. The two noteworthy depressions in its surface, the Valley of Mexico and Bolson de Mapini, once contained large bodies of water, of which only small lakes and marshy lagoons now remain.

The highest part of this great plateau is to be found in the States of Mexico and Puebla, where the general elevation is about 8 000 ft. Southward, the slope is broken into small basins and terraces by transverse ranges, and is comparatively abrupt. Northward, the slope is gentle, and is broken by several transverse ridges. At Ciudad Juarez (adjoining El Paso, Tex.), on the northern frontier, the elevation is 3 600 ft., which shows a slope of only 4½ ft. to the mile.

The mountain ranges which form part of the great Mexican Plateau consist of two marginal chains known as the Sierra Madre Occidental, on the west, the Sierra Madre Oriental, on the east, and a broken, weakly-defined chain of transverse ranges and ridges between the Eighteenth and Twentieth Parallels, known as the Cordillera de Anahuac. The Sierra Madre Oriental consists of a broken chain of ranges extending along the eastern margin of the plateau from the great bend in the Rio Grande southeastward to about the Nineteenth Parallel. In the north these ranges are low, and offer no great impediment to railroad building; south of Tampico, however, they are concentrated in a single lofty range. This range extends southeastward along the western frontier of the State of Vera Cruz and includes the snow-capped cone of Orizaba, or Citlapetl (18 209 ft.), and the Cofre de Perote, or Nanchampopetl (13 419 ft.). The eastern slopes are abrupt and difficult, and form a serious impediment to communication with the coast. Rising from the open plateau half way between this range and the City of Mexico is the isolated cone of Malinche, or Malintzin (14 636 ft.). Some 20 miles southeast of the City of Mexico are the gigantic snow-clad volcanoes of Popocatepetl (Smoking Mountain) and Ixtaccihuatl (White Woman), 17 888 and 17 343 ft., respectively.

The lowland or *tierra caliente* region, which lies between the Sierras and the coast on both sides of Mexico, consists of a sandy zone of varying width along the shore line, which is practically a tide-water plain broken by inland channels and lagoons, and a higher belt of land,

rising to an elevation of about 3 000 ft., formed in great part by the débris from the neighboring mountain slopes.

The peculiar surface formation of Mexico, a high plateau shut in by mountain barriers with a narrow lowland region between it and the coast, does not permit of the development of large river basins. The hydrography of Mexico, therefore, is of the simplest description, a number of small streams flowing from the plateau or mountain slopes eastward to the Gulf of Mexico and westward to the Pacific.

4.—Explorations and Surveys.

On December 2d, 1857, Don Antonio Escandon, in consultation with Gen. Manuel Robles, who represented the Mexican Government at Washington, engaged the services of Andrew Talcott, former Captain in the Corps of Engineers, United States Army, to make surveys for a railroad from Vera Cruz to the City of Mexico.

Immediately after his appointment Capt. Talcott organized a corps of engineers in the City of New York, which sailed from New Orleans, La., on January 1st, 1858, and landed in Vera Cruz on January 4th. The members of this corps were: Capt. Andrew Talcott, Chief Engineer; Andrew B. Gray, Quartermaster and Commissary; J. S. Stoddart, Richard Page, and L. Van Wyck, Assistant Engineers.

First Brigade.—M. E. Lyons, Chief Engineer; E. M. Richards, R. F. Stack, T. W. Sargent, W. W. Dechert, A. C. Hall, E. McLenegan, J. Radnich, James J. Arnold, and George E. Clymer.

Second Brigade.—R. B. Gorsuch, Chief Engineer; D. S. Crosby, W. R. Eastman, A. Van Burcke, James Emmet, T. A. Emmet, H. G. Webber, L. B. Ward, O. W. Boynton, Alfred Delano, and D. M. Gorsuch.

Exploring Party.—Sidney Coolidge, Chief; T. M. R. Talcott, Charles Miller, C. A. Wolf, and George R. Talcott.

Of the foregoing, Messrs. T. M. R. Talcott (son), of Richmond, Va., and W. R. Eastman, of Albany, N. Y., are still living.

On his arrival Capt. Talcott found the country in a state of turmoil; for 3 years Mexico was a prey to civil war between two rival governments, the Republicans at Vera Cruz under Juarez, and the Reactionaries at the Capital.

With a view to an examination of the two principal routes from Vera Cruz to the table-land, one *via* Orizaba, and the other *via* Jalapa, two brigades of engineers had been organized before leaving the United States, as previously indicated.

Viewed from the ocean side, in the State of Vera Cruz, the almost perpendicular rocky walls, rising to a height of 10 000 ft. and more above ocean level, seemed to preclude the possibility of constructing a railroad from the lowlands to the plateau, but this was the problem which confronted Capt. Talcott and which he was called on to solve.

Comparative quiet having set in, the surveys of these two routes progressed favorably until the Constitutional President, Benito Juarez, who had escaped by the Pacific Coast, arrived at Vera Cruz, and military operations became more general, with the inevitable *pronunciamentos* which unsettled conditions wherever they occurred and made it difficult for the engineering parties to secure proper passports from recognized authorities to enable them to carry on their work without interruption and accomplish it at reasonable cost.

The cost of the surveys was being paid by Don Manuel Escandon, and they would not have been undertaken at this time if he had foreseen the breaking out of a revolution; but he was not willing to abandon them until it was determined that it was practicable to build a railway from Vera Cruz to the City of Mexico, and ascertain approximately what it would cost.

For these reasons, it was decided to reduce the force of engineers, and as the line *via* Orizaba was more advanced and conditions were less threatening on that route, it was deemed best to discontinue the survey *via* Jalapa until some future time, and concentrate the engineering force which was retained, in order to complete the survey of the Orizaba route as quickly as possible.

Previous to the arrival of Capt. Talcott, the Jalapa route had been examined by Don Pascual Almazan, a Mexican engineer. Subsequently, this route was examined (reconnoissance) by one of Capt. Talcott's assistants, Mr. E. M. Richards, whose report was unfavorable.

The Orizaba route was preferred, first, because it passed through towns of greater importance and a more cultivated and productive country; and second, because, notwithstanding the fact that it required great and difficult works, the line presented greater facilities than that by Jalapa, where the large number of ravines and the harder

nature of the soil would have required a much heavier outlay, without meeting the resources offered by the Orizaba route.

By the terms of the concession granted to Señor Escandon by the Mexican Government, surveys for two routes were required, one carrying the main line through Puebla, the other connecting Puebla with the main line by a branch. This was accomplished in the first case by running a line from the Maltrata Pass to Puebla, and thence to Apizaco in the Llanos de Apam, and, in the second case, by running a northerly line from the same point direct to Apizaco, the surveyed line from Puebla to Apizaco thus serving either as a part of the main line or as a branch line, as would be decided later.

On the completion of the surveys, in October, 1858, the engineers returned to the United States, and, under date of March 7th, 1859, Capt. Talcott forwarded to Señor Don Antonio Escandon his report "of the cost of constructing, maintaining, and operating a railway from Vera Cruz to the City of Mexico."

The receipt of this report was acknowledged by Señor Don Antonio Escandon from Mexico, under date of April 18th, 1859, and the report was also referred to in a letter from Tucabaya, dated May 19th, 1859, by Señor Don Manuel Escandon, a brother. In the former, Señor Escandon said:

"I consider it as a positive fatality that the unfortunate state of Mexico, with the civil war which now exists, prevents me for the moment from continuing the prosecution of a work which would most certainly become the solution of the real peace and prosperity of my country."

The report, as submitted by Capt. Talcott, is as follows.

5.—Report of Captain Andrew Talcott.

"Sr. Don Antonio Escandon,

"President of the M. & P. R. R. Co.

"Sir: On the 4th December, 1857, you engaged my services as Chief Engineer of your railway enterprise. It was at that time stipulated and agreed that the surveys between the port of Vera Cruz and the City of Mexico should be prosecuted vigorously and with a view to such a development of the topography of the country as would enable you to select the route of your proposed railway, define the same by maps, profiles, and specifications, and submit them to the Supreme Government for approval on or before the 31st of April following. This was one of the conditions set forth in the Decree of 31st of

August, 1857, granting you the exclusive privilege of connecting by a railway, the Capital of the Republic with the commercial emporiums of the Gulf and Pacific Coasts, Vera Cruz and Acapulco, or any other port, that might be deemed more eligible for the rapidly increasing commerce of the Pacific Coast.

"The time allowed was short, for the labor to be performed, *viz.*, to organize and equip a suitable Corps of Engineers, transport them to Vera Cruz, and thence with their instruments and camp equipage to the mountain passes, develop the most practicable of them, and from it to extend the surveys to Vera Cruz, and to the Capital, by lines of more than 260 miles in extent, and partly over ground exceedingly difficult and covered with the thick undergrowth, verdure, and vines of the tropics.

"A Corps of Engineers fully equipped and prepared for the service was landed at Vera Cruz on the 4th of January, but in consequence of the robbery and loss of the mail by the preceding packet from the United States, no arrangements had been made there for the transportation of the Corps; its movements were therefore so much delayed that over four weeks had elapsed before the Engineers were fairly at work surveying and exploring the mountain passes. During this interval a revolution had been in progress, and had resulted in the overthrow of the Constitutional Government at the Capital. It was the opinion of the best informed in Mexico that the struggle for power between the two contending parties would not soon be terminated by the triumph of either, and that the surveys could not be vigorously and successfully prosecuted during the then existing state of affairs. The obligation was no longer in force requiring the route to be defined and specifications filed before the 1st of May, as that had been suspended by the 26th Article of the Decree providing for such a contingency as had actually occurred. As only a small Corps could be advantageously employed for any length of time, whilst the Civil War lasted, a large reduction of the Corps was decided upon and effected, so that two-thirds left the country by the packet which sailed on the 7th of March from Vera Cruz.

"One important fact had however been ascertained, by the explorations and surveys previous to the reduction of the Corps, enough had been done to demonstrate that a railway could be constructed from Vera Cruz to the table-lands on grades and curves, susceptible of being worked by locomotive power, economically, and that the cost of such road would not be excessive. The survey had been confined to the route *via* Orizaba, but reconnoissances however had been made *via* Perote by Mr. Almazan, previous to the arrival of the Corps, and afterward verified by one of the assistants, Mr. Richards, whose reports were not favorable to the Northern route, but it had not been decided that a railway was impracticable; that the surveys were not made

with the same minuteness by the Actopan Valley, was partly from the want of time, to which may be added the civil strife which was waging in that section from March until the Corps left the Republic in October; but for the above mentioned causes this route would have been surveyed with sufficient minuteness to make a comparison of it with that *via* Orizaba. The examinations made were, however, sufficient to show that it is greatly inferior to the Orizaba route, though not minute enough to allow comparisons to be made of length, grades, curves, and cost of construction.

"It has been stated that a road by Orizaba can be economically worked by locomotive power, nevertheless it will be a road of high grades. The route surveyed reaches the table-land by a line of 111 miles, and attains at that point an elevation of 8 101 English feet, 4 000 of which must be overcome west of Orizaba, and where the summit is, in a right line, distant less than 16 miles.

"The face of the country is, however, favorable for developing and by the survey the distance exceeds 28 miles, and can be increased several miles if desirable. The question therefore arises: Is it desirable for the purpose of reducing grade to further develop the road? This is strictly a question of economy, and can be solved with much certainty. The locomotive and train must ascend the 4 000 ft., the line may be lengthened, and the cost of construction thereby increased, and an additional weight of train would be the consequence. There is a point, however, where the cost of working the road would be a minimum, this could be fixed with perfect precision, if the adhesion of the engine to the rail was constant, but it is not so, sometimes it is as low as 14% of the insistent weight, and it goes as high as 30 per cent. The grade most economical, so far as working power is concerned, changes as the adhesion changes. When the adhesion equals 30% a grade of $2\frac{9}{10}$ ft. per hundred would be worked with the least expense of power, at 20% $2\frac{3}{10}$ ft. per hundred; and 14% $1\frac{9}{10}$ ft. per hundred. These are the grades for up traffic only. Should the grade be increased to $4\frac{6}{10}$ ft. per hundred, the cost of power would only be increased $7\frac{2}{10}$%, with the adhesion of $\frac{1}{5}$th or 20 per cent. As the adhesion diminishes, the cost of working increases. (See Note *A*.)

"Thus far we have considered the expenses of but one item of railway transportation, that of power. The maintenance of way is another item generally as large on ordinary roads as the cost of power. Every mile added to lessen the grade increases the cost of keeping the road in repair, and the time required to travel over it. The greatest economy lies therefore in grades exceeding those that require the minimum power, even if the cost of construction be not thereby diminished, but under ordinary circumstances with higher grades the line would be shorter, and the reduction of cost greater in proportion, than the reduction of length. If a mountain line be 10 miles long,

with grades of 4 ft. in a hundred, it is probable that with grades of $4\frac{6}{10}$ ft. in a hundred, there will be a reduction of at least 10% in the length and more than that in the cost. The saving of the cost of this mile, and of the keeping of it in repair, will fully compensate for the additional power required for working the increased declivity. But the minimum cost of constructing, maintenance, and workings will depend on the amount of traffic, as well as the cost of power and maintenance, and from my investigations it appears that the most economical grade will not vary much from $4\frac{6}{10}$ ft. in the hundred, and this may be safely assumed as the maximum grade of the mountain road above Orizaba. The location which has been made with a ruling grade of 4 ft. in a hundred may therefore be revised, and a considerable saving thereby made, and curves of 400 ft. radii substituted for those of 350 ft., which in the experimental location were admitted on the mountain line. In all other portions the curvature is unobjectionable.

"By the terms of the grant, Vera Cruz, situated on the Gulf of Mexico in Latitude 19° 11′ 52″, was to be the eastern terminus of the railway. From this city a railway had already been constructed for about 14 miles, the title to which having been transferred to you, it was a matter of some importance to include as much of this road in your proposed line as could be advantageously used. It was found that about 9 miles could be used by the route *via* Orizaba, and an experimental location was made from this point to Cordoba, a distance of 25 leagues from Vera Cruz.

"This was as far as it was expected to extend the surveys at the time the Corps of Engineers was reduced in March, but this part of the country having become in a measure tranquil, and the labors of the Engineers uninterrupted, the experimental lines were continued up the country to the summit of Maltrata Pass and thence by Cuesta Blanca and Acacingo to Puebla, and by the Plains of Apam to Mexico. A second line was also run, branching from the first near the summit, and passing by a more northern route near Nopalucan and Huamantla, and intersecting the first line again some distance northwest of Tlaxcala. All these lines have been mapped and profiled, and made the basis of an approximate estimate of cost of the railway from Vera Cruz to the City of Mexico. It should be borne in mind that these are first experimental lines, and have not been perfected or revised into an 'Experimental Location' but still they afford very good data for an approximate estimate. It cannot be doubted that in many places ground better adapted may be found, and this is most especially the case going from Puebla to the Plains of Apam. In some places the cost may be considerably reduced, or better grades secured at the same cost. Grades have been laid down on the profiles of all the surveyed lines, and the quantities of excavation and embankment required to form a roadbed 16 ft. wide, with suitable slopes, have been carefully

calculated and entered upon the profiles, from which tables of quantities have been compiled which will be found appended hereto (marked 1).

"It appears from these surveys, that a railway *via* Puebla to the City of Mexico is $22\frac{8}{10}$ miles longer, than *via* Nopalucan, and the cost will be much greater; but Puebla is a city of too much importance to be excluded from such an enterprise, and must be served by a branch, if the more northern route is adopted. Which of the two projects should have the preference involves numerous considerations, and one very important one is the amount of direct traffic between Vera Cruz and Mexico. The greater it is, the stronger will be the objection to transporting it *via* Puebla, which not only increases the distance of carriage, but descends into the Palmar Valley 1 002 ft., to be again carried up to the northern line 824 ft.

"There are, however, many considerations to be urged in favor of the route *via* Puebla, as the trunk line, but it is not necessary to decide on the route at present, and better light may be hereafter obtained to guide your decision.

"By Puebla your road will be $289\frac{2}{10}$ miles in length, by the northern line $266\frac{4}{10}$ miles, and as a branch to Puebla will be $29\frac{6}{10}$ miles, it will require 296 miles of road, or $6\frac{8}{10}$ miles more than by Puebla.

"The cost of completing the several lines is exhibited in the following table:

"TABLE 1.—Estimate of the Cost of Completing a Railway from Vera Cruz *via* Orizaba, and Puebla, to the City of Mexico.

" Locality.	Length of line, in miles.	Roadway.	Track.	Structures.	Totals.
Vera Cruz to Cordoba	67.8	$1 625 000	$882 000	$169 000	$2 676 000
Cordoba to Orizaba	16.3	949 000	252 000	87 000	1 288 000
Orizaba to Maltrata Summit	28.1	1 392 000	420 000	18 000	1 830 000
Maltrata Summit to Puebla	60.2	681 000	903 000	74 000	1 758 000
Puebla to Junction	29.6	599 000	448 000	18 000	1 065 000
Junction to Mexico	87.1	358 000	1 232 000	219 000	1 809 000
Totals	289.1	$5 604 000	*$4 137 000	*$585 000	$10 426 000

Total cost of roadway	$10 426 000
Total cost of equipment	*1 260 000
Amount already expended on San Juan and Guadalupe Railways and on surveys	1 400 000
Total for roadway and equipment	13 086 000
No data for estimating { General management, real estate, land damages, etc.	

* See Notes *B*, *C*, and *D* for details of these estimates.

"As the road progresses, furniture to the amount of $2 500 per mile of road in operation will be required, and more as the road lengthens and business increases. On its completion, no less than $400 000 will

be required to equip each division, of which there must be three, and in a few years it may be expected that a further quantity will be required, all of which must, however, depend on the amount of business.

"By the northern route, the cost of main line will be less by	$1 364 000
"The Puebla Branch and structures at Puebla are estimated at	1 278 000
"Difference of cost against Puebla Line only..	$86 000

"Full details of the work are exhibited in the accompanying papers, to which reference is respectfully requested.

"TABLE 2.—QUANTITY OF EXCAVATION, EMBANKMENT, MASONRY AND TUNNELING ON THE SEVERAL PORTIONS OF THE ROAD FROM TEJERIA, *via* ORIZABA AND PUEBLA TO THE JUNCTION OF THE GUADALUPE RAILROAD.

" Portions of line.	Length of line, in miles.	Excavation, in cubic yards.	Embankment, in cubic yards.	Masonry, in cubic yards.	Tunneling, in linear feet.
Tejeria to Cordoba........	58.4	881 036	1 033 690	50 095	840
Cordoba to Orizaba.......	16.3	657 194	438 822	40 822	700
Orizaba to Maltrata Summit.....................	28.1	429 117	607 806	179 115	660
Maltrata Summit to Puebla..................	60.2	414 519	757 685	16 289	....
Puebla to Junction........	29.6	174 582	449 677	33 368	....
Junction to junction with Guadalupe Railroad....	83.9	419 015	378 051	5 114	....
Totals................	276.5	2 975 463	3 665 731	324 803	2 200

"The country traversed by the railway will afford all the material required for its construction except iron. Timber, though not abundant throughout the line, is sufficiently so in some parts of it for the supply of all. In the estimates, suitable provision has been made for its transportation, as well as for rendering it more enduring by some of the processes usually resorted to for that purpose where timber is expensive. An interesting commentary from Hugo Fink, Esq., on the timber of Mexico is appended to this report (marked 2).

"There are some features in this line of road that require particular notice, for though for three-fourths of the distance it presents lines of easy grades and open curves, requiring few expensive bridges, cuts, or embankments, there are exceptions.

"The first of these occurs at the Jamapa River, at the village of Soledad, where a bridge is required 650 ft. in length, and the grade

line is 100 ft. above the river at its low stage. I have estimated the cost of such a structure, all of masonry, at $216 000. No difficulty will be encountered in obtaining foundations for the piers and abutments. The next bridge of any magnitude will be over the Canalita Creek, and here also a permanent structure of masonry is estimated for at the cost of $90 000. Limestone of superior quality abounds in this vicinity, it is the first we find 'in place' on the line above Vera Cruz. Inexhaustible quarries may be opened for the supply of the Vera Cruz market, for building purposes, including moles, breakwaters, forts, etc., and the quality is so good that vessels lacking return cargoes may find a profit by ballasting with it for some ports. The next expensive structure will be for crossing the Atoyac River where it breaks through the Chiquihuite Mountains. Here a deep ravine has been cut in the solid limestone rock, the great depth and precipitous sides of which present formidable obstacles to a railway. To span this chasm on the present line of location will require a bridge 300 ft. in length at an elevation of 250 ft. above the stream, but by adopting a lower line, both the great height and the length of the structure may be considerably diminished. At all events, it would seem to be a point where an iron bridge may be used to advantage, and in the estimate I have allowed $275 000 for the construction of one. (See Note *F.*)

"The crossing of the Rio Seco will require 300 ft. of bridging, at an elevation of 55 ft., which is estimated to cost $50 000. Between Cordoba and Orizaba is the deep *barranca* which is crossed by the public road near the village of Fortin, by which name it is known to many, though to others as the Barranca de Metlac. The point at which the fewest obstacles are presented to a crossing for a railway appears to be about 2¼ miles north of Fortin, where it was found to be 1 085 ft. wide at the grade line, which was 346 ft. above the stream. The rock on the west side slopes about 45°, the eastern slope being still steeper, consequently the span of the bridge can be but little reduced, except at an enormous cost for embankments. It is a fine site for a suspension bridge, and if that kind of structure was not very objectionable on railroad lines, I should recommend the construction of one at this point as the cheapest method by which this ravine can be crossed. A better, but much more expensive plan would be an 'Iron Tubular Bridge' of two or three spans, resting on piers formed of cast-iron tubes, which would be cheaper than masonry.

"No detailed plan or estimate for this viaduct has been prepared, but from such data as I have been able to collect, it may be estimated at $1 200 000. (See Note *F.*) I do not recommend its construction at an early day, for we find that, by running the line down the eastern slopes of the *barranca,* it may be crossed on a bridge 600 ft. in length, and at an elevation of 140 ft., and where it is sufficiently wide to turn

the road on a curve of 400 ft. radius in order to ascend the opposite side. The road is thus brought back to the point of the high crossing and without the admission of grades higher than 158 ft. per mile, which is not objectionable, considering that the same curvature and much higher grades are admitted above Orizaba.

"This change will reduce the cost to $400 000. After passing the *barranca* no expensive work is required, except on the 9 miles of the mountain line near Maltrata Summit and 3 miles in descending into the Acacingo Valley at Cuesta Blanca.

"In estimating the cost of work, about 50% has been added to the prices usually paid in the United States, where free labor is employed, and the same percentage, also, has been added to the cost of equipment, to cover the transportation, or the extra cost of such portions as may be manufactured in Mexico.

"A most liberal estimate has been made of the cost of track, the rails to weigh 60 lb. per yd., of a superior quality of iron, with fished joints, on sleepers, either creosoted or cyanized, to give them greater durability; this is deemed indispensable on account of the scarcity of timber in Mexico, as well as the effect of the climate upon it when not so prepared, especially in the hot and temperate portions of the country.

"The cost of railroad work in Mexico, I believe, will, after the first year or two, be very little greater than in the United States. Labor at the commencement of operations must be taken to Mexico at the expense of the parties doing the work; but when a demand for it, with constant and steady employment at fair wages, is established and generally known in Europe, and in the United States, it is but reasonable to suppose that emigration will move in that direction to the full extent that employment can be given to it; and the cost of work should then be very little, if any, greater than in the United States. Except in the hot country, a man can labor as many hours, and with as much effect, as in any other country; subsistence and clothing are not more expensive, if as much so, as in New York or Ohio. It would not, therefore, seem to be judicious to make very large contracts at the commencement of the work; otherwise all the reduction in the cost of labor would go to the benefit of the contractor, who as a first experiment would calculate on liberal prices, and insist on a large margin to meet contingencies, the most of which a slight advance will cover, after allowing for time required for traveling and cost of transportation of such labor as must be engaged in foreign countries. The instability of the Government would seem to be the greatest obstacle to cheapening labor in Mexico, but that I conclude is a risk to be borne by the Company, and not by the Contractor.

"The following is believed to be a fair estimate of the probable cost of maintaining and operating 266½ miles of railway in Mexico, with

the number of trains that may from time to time be required for its business:

"With 2 daily trains	$365 000	per annum.
" 4 " "	580 000	" "
" 6 " "	798 000	" "
" 8 " "	1 015 000	" "

"For the details of this estimate see note (marked *E*) where the number of trains for passengers and freight has been assumed to be equal, or in some cases mixed for passengers and freight.

"The data are not in my possession for estimating the present or prospective amount of trade and travel over your road.

"The foregoing estimate of the cost of constructing, maintaining, and operating it, with from 2 to 8 daily trains, each capable of conveying 200 passengers, their baggage, and the mails, in 16 hours, or 200 tons of freight in 36 hours, from Vera Cruz to the City of Mexico, will, with that of your statistical commission, afford sufficient data for investigating the financial prospects of the enterprise. From the very imperfect observations made of the amount of business, during the progress of the surveys, though a civil war was then raging, I should estimate that more than Three Million of Dollars was paid annually for the transportation, that can be better done by a railway for one-quarter of that amount.

"The maps and barometrical profiles prepared under your authority by Mr. Almazan, and handed to me a few days after the landing of the party in Mexico, were of great value, as they enabled me to place the parties on the lines of greatest promise before a reconnaissance could be made in person. To the Engineers employed in the enterprise I am under great obligations for the prompt and satisfactory execution of all the duties assigned them, whether in the field, or in the office.

"All of which is respectfully submitted.

"Andrew Talcott,
"New York, March 7th, 1859." *"Chief Engineer."*

6.—Summaries of Special Reports, *A, B, C, D, E,* and *F.*

In addition to his general reports, Capt. Talcott also submitted the following special reports or notes:

A.—Effective Working of Engines on Grades.—This is a study of the tractive power of locomotives to determine the most economical grades for the Maltrata Incline. It was assumed that an engine weighing 25 tons, with a tender of 20 tons, when charged with fuel and water, could take a train of 130 tons from Vera Cruz to Orizaba; thence to the Maltrata Summit an additional engine of 25 tons, constructed for that service, would be required to assist in reaching the summit.

B.—Estimate of Cost of One Mile of Track and Keeping it in Repair until Opening of Section for General Traffic.—This contains remarks on the comparative economy of best quality with inferior rails; extract from a report of the President of the Philadelphia, Wilmington and Baltimore Railroad Company, in 1858, on the subject of rails; and extracts from a report on European railways, by Colburn and Holley, relative to rails. At the time of making this report (1858-59), merchantable rails, or such as were ordinarily manufactured for the American market, could be purchased for from $33 to $41 per ton. An estimate of $70 per ton was made for rails delivered in Vera Cruz, which allowed considerable margin for changes in the market, as well as for better rails than those ordinarily made for the United States, it being assumed that good rails should last from 12 to 16 years. The experience of the Philadelphia, Wilmington and Baltimore, Philadelphia and Reading, and also of English railroads, was cited as to weight and manufacture of rails, and makes interesting reading, showing that rail troubles were not unknown at that time.

The cost of 1 mile of track for the Mexico and Pacific Railroad was assumed at $14 000, the weight of rail per yard being 60 lb., although it is not specifically stated, but is inferred from the quantity given per mile.

This report contained the following estimate of the cost of 1 mile of track and of keeping it in repair until the opening of the section for general traffic:

2 440 cu. yd. of ballast at 60 cents	$1 464
2 750 cross-ties at 85 cents	2 337
95 tons of rails, extra quality, landed at Vera Cruz, at $70	6 650
6 tons of fish-plates at $80	480
1¼ tons of bolts and nuts at $120	150
8 000 lb. of spikes	400
Dressing surface of roadway and preparing for the track	150
Transporting materials for 1 mile (average)	1 250
Laying 1 mile of track	600
Extra cost of switches per mile	125
Total cost of 1 mile	$13 606

Brought forward	$13 606
For repairs of slopes during construction, and keeping track in repair	394
Total cost of 1 mile with repairs	$14 000
Cost of rails, spikes, plates, etc	$7 805
Cost of cross-ties	2 337
Total cost of materials	$10 142
Ballast	1 464
Labor and transportation	2 394
Total cost of 1 mile	$14 000
Cost of iron for 280 miles	$2 185 400

C.—Buildings and Structures for Stations.—The sum of $585 000 was estimated for the buildings and structures required for the principal stations of the railway from Vera Cruz *via* Orizaba and Puebla to the City of Mexico, and for way stations. Water stations, engine stalls, and repair shops were included in this item. The largest proposed expenditure was at the City of Mexico, where the sum allowed was $237 000.

D.—Equipment.—It was not thought probable that more than one passenger train daily would be required for many years, except, perhaps, on the arrival of steam packets at Vera Cruz, when an extra might be needed, and that was provided for in the following estimate. It was also estimated that one freight train could take from Vera Cruz, over any grade east of Orizaba, 160 tons of grass, or 100 tons of freight, which is equivalent to 30 000 tons per annum, a quantity greater by one-half than the present imports at Vera Cruz (1858), estimated at 20 000 tons annually. It was thought probable, however, that on the opening of the entire road, from Vera Cruz to the City of Mexico, a second freight train daily would be needed. The sum of $1 260 000 was allowed for equipment, detailed as follows:

54 engines	at $8 000 =	$432 000
36 passenger cars	" 2 500 =	90 000
18 mail and baggage cars	" 1 500 =	27 000
90 house cars	" 700 =	63 000
60 platform cars	" 500 =	30 000
Carried forward		$642 000

Brought forward..................			$642 000
30 gravel cars.......................at	400	=	12 000
30 crank cars......................"	150	=	4 500
30 hand cars......................."	50	=	1 500
Cost in the United States...........		=	$660 000
Add 50% for cost at Vera Cruz.........		=	330 000
			$990 000
6 turn-tables, stationary engines, etc..........		=	210 000
4 mountain engines.........................		=	60 000
Total........................			$1 260 000

E.—Cost of Maintaining and Operating One Mile of Railroad in Mexico.—This cost was estimated to range from $365 000 per annum for two daily trains to $1 015 000 for eight trains.

F.—Iron Girder Bridges.—Letter from William Fairbairn.—In Capt. Talcott's report, mention is made of two high river crossings, necessitating bold and costly bridges. One, the crossing of the Rio Atoyac, between Paso del Macho (Mule Pass) and Cordoba, the other, the crossing of the Barranca de Metlac, between Cordoba and Orizaba, both deep chasms in which flow torrential streams during the rainy season, but which are dry the remainder of the year.

The Rio Atoyac rises in the Chiquihuite Mountains, and flowing in a southeasterly direction, joins the Rios Chiquihuite and de San Alejo, which empty into the Rio Jamapa. The proposed railroad crossing was 300 ft. long and 300 ft. high.

The Rio de Metlac rises among the fissures of the Pico de Orizaba, and, after a sinuous course, is lost in the plains adjoining the slopes of the Cacalote and Chiquihuite Mountains. It is crossed by the Vera Cruz-Mexico City highway near the Village of Fortin. This crossing was made by a series of zigzags, down one side and up the other, the opposite banks being united by a stone bridge. At the place of the crossing of the original line of survey, this ravine is more than 1 000 ft. wide and about 350 ft. deep below the adjoining plains of Cordoba on the east and those of Orizaba on the west.

At first a suspension bridge was suggested for the railroad crossing, but this design was afterward changed to an iron tubular

bridge after consultation with Mr. William Fairbairn, the well-known English engineer of the last century.

The following extract from the report of Capt. Talcott relative to the cost of tubular bridges for the two crossings mentioned is of interest in these days of cantilever structures:

From the letter of Mr. Fairbairn, * * * it appears that iron plates for a tubular bridge of 317 ft. span will cost in Liverpool £15 333 sterling, and that 1 034 ft., that is, one span of 400 ft. and two of 317 ft., would cost £57 166. One pier of masonry in the Barranca de Metlac, 346 ft. in height, would cost $225 000. One of cast-iron columns under a permanent compression of 1 ton per sq. in. would require 1 875 sq. in. of iron at the bottom, and 1 500 sq. in. at the top, equal to 2 tons per ft. of height, or 850 tons of cast-iron columns, to which should be added for the connection 150 tons, or, in round numbers, for one pier of cast iron, 1 000 tons, mostly of cast iron, which I estimate can be purchased in Liverpool, all fitted, for £9 000. For two piers, the cost of each would be about one-sixth less, or for the two £15 000. From these data I infer that my estimate of $1 200 000 for bridging the *barranca* is a very liberal one.

"MANCHESTER, NOV. 26TH, 1858.

"SIR.—In reply to your letter of 3rd I have to refer you to the annexed pen sketch of the bridges I would propose for crossing the ravines or rivers, on the line of railway between Vera Cruz and Mexico. I assume that, in crossing these valleys, it will be a single line of rails, and should this be the case the following will enable you to form a correct idea of the cost of the ironwork forming the superstructure. You will observe that I have divided the wide ravine of 950 ft. into three spaces, as being much cheaper, but I have given you the cost both ways, in order that you may draw your own conclusions as to which is the most eligible, or most expedient to be adopted.

"For the 400-ft. span we have as under:

"Tubular bridge of 400 ft. span, prepared in sections and delivered in Liverpool ready for re-construction including all the plates, bolts, rivets and expansion apparatus, the whole complete, about 1 250 tons....	£26 500
"Also, tubular bridge in three spans of 317 ft. each, collectively 950 ft. = 2 150 tons and delivered, as above..	46 000
"Total for both bridges delivered in Liverpool...........	£72 500

"To make the latter bridge of two spans, the weight will be 3 000 tons, and will cost with expansion apparatus, etc., from £65 000 to £66 000, making a difference of £20 000 in the cost of a bridge of two spans, compared with one of three spans.

"The computed strength of these tubes will be 8 tons per lin. ft., or 3 200 tons for the 400-ft. spans, and 7 600 tons for the three spans. The piers, *A A*, may be made of iron if you prefer it. For all your commands "I am,

"Dear Sir,

"Yours sincerely,

"M. E. LYONS, ESQ., C. E." "WM. FAIRBAIRN."

Although Capt. Talcott did not recommend the immediate construction of the high Metlac Bridge, and, as an alternative, proposed a lower bridge by running the line down along the eastern side of the *barranca* to a suitable crossing and then ascending along the opposite side, the original plan of building the high bridge was adhered to for some time.

On April 26th, 1866, nearly 8 years after the report of William Fairbairn, the cornerstone was laid with appropriate ceremonies, the occasion being referred to as the "Grand Demonstration, Cornerstone Laying of Big Bridge", at which were present Señor Don Antonio Escandon, the Perfect of Orizaba, Engineers William Lloyd, Sebastian Wimmer, and others. The "demonstration" was followed by a dinner and dance.

Following this function, a pronounced earthquake occurred on May 10th, 1866, exactly 2 weeks after, which phenomenon caused the immediate abandonment of the original project, fearing for such a structure the disastrous consequences of a repetition.

The alternative line was then substituted and made a part of the location between Cordoba and Orizaba, which was run in 1866 by Gen. H. T. Douglas, an assistant of Capt. Talcott.

Under date of June 28th, 1871, Mr. William Cross Buchanan, who at that time was Chief Engineer of the Mexican Railway, as then constituted, approved this change and designed the structure crossing the *barranca,* known as the Metlac Viaduct. It consists of nine spans, each of 50 ft., and is built on a curve with a radius of 325 ft. (17° 40′), with the track at an elevation of 80 ft. above the level of the river which gives the name to the ravine. The bridge was constructed at the famous ironworks at Crumlin, England. Each pier was composed of eight cast-iron columns, of which the inner four were vertical, and the outer four had an inclination or batter of 1 in 8. These piers carried the plate girders, and these, in turn, carried the track, cross-ties, and rails.

7.—Maps, Profiles, Plans, and Estimates.

Accompanying Capt. Talcott's report was a general map showing the entire line, on a scale of 1 in. = 5 miles. There were also detailed maps of the nine divisions into which the line was divided, generally between the large cities as, Vera Cruz-Soledad; Soledad-Cordoba; Cordoba-Orizaba; these maps were drawn to a scale of 1 in. = 3 000 ft., except that for the Orizaba-Maltrata Summit Division, which was 1 in. = 1 000 ft. There was also a map of the public road from Mexico to San Martin on the larger scale.

In addition, there were eight profiles of the various divisions, and plans of proposed buildings and bridges.

There was also a detailed estimate by mile sections of the cost of graduation and masonry, summarized by divisions, the names of these being given in Capt. Talcott's report in Tables 1 and 2, under the headings, "Locality and Portions of Line."

The graduation was divided into three classes: excavation, hauled into embankment, and borrowed material. On the Tejeria-Cordoba Division, 58¼ miles, the assumed unit prices per cubic yard for excavation were, between Tejeria and Purga, about 10 miles, 25 cents; between Purga and Atoyac, along the foot-hills of the Chiquihuite Mountains, about 31 miles, 50 cents; between Atoyac and east of Paraje Nuevo, about 8 miles, in the Chiquihuite Mountains, $1.50; the material at the latter place being limestone rock. On the remaining 8 miles, the excavation price was generally 30 cents, although there was some at 50 cents and some at $1.50. For "hauled into embankment" (extra haul), the assumed price ranged from 3 to 27 cents; and for "borrowed material", 25 and 30 cents. Bridge masonry was estimated at $10, ordinary masonry at $8, and dry masonry at $4. There were two tunnels on this Division, respectively, 460 and 380 ft. long, a total of 840 ft., which were estimated at $75 per lin. ft. The cost per mile ranged from $926.80 for Section No. 8 to $78 360.58 for Section No. 57, where no tunneling or high viaducts occurred. The Atoyac Bridge Section, No. 45, was estimated to cost $304 268.15, of which $275 000 was for the bridge. About one-half the estimated cost of this Division, more than $832 000, was concentrated at four points, the crossings of the Jamapa, Chiquihuite, Atoyac, and Seco Rivers, making the average cost of 54 miles about $13 000 per mile, exclusive of track.

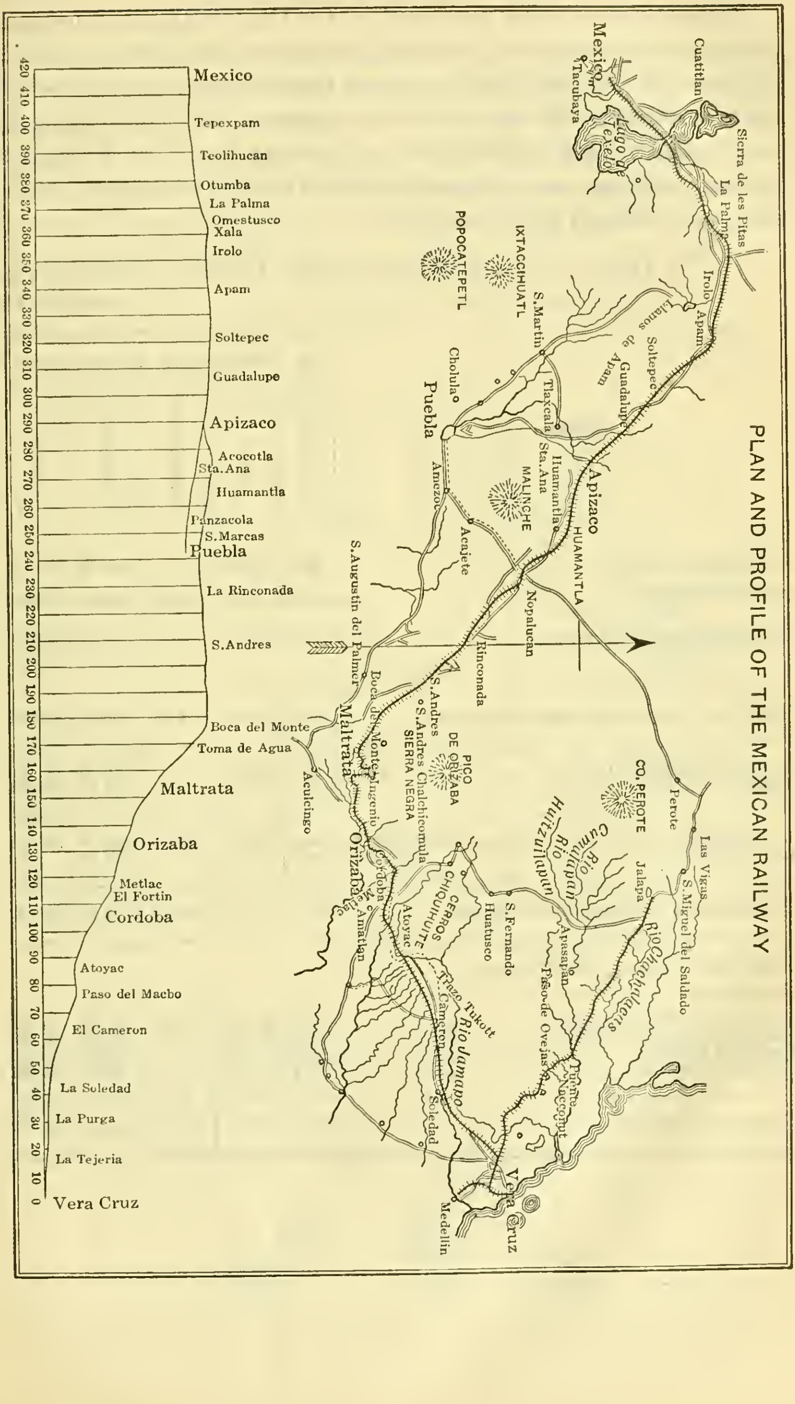

PLAN AND PROFILE OF THE MEXICAN RAILWAY
420 410 400 390 380 370 360 350 340 330 320 310 300 290 280 270 260 250 240 230 220 210 200 190 180 170 160 150 140 130 120 110 100 90 80 70 60 50 40 30 20 10 0
Mexico
Tepexpam
Teolihucan
Otumba
La Palma
Omestusco
Xala
Irolo
Apam
Soltepec
Guadalupe
Apizaco
Acocotla
Sta. Ana
Huamantla
Panzacola
S.Marcas
Puebla
La Rinconada
S.Andres
Boca del Monte
Toma de Agua
Maltrata
Orizaba
Metlac
El Fortin
Cordoba
Atoyac
Paso del Macho
El Cameron
La Soledad
La Purga
La Tejeria
Vera Cruz
Cuatitlan
Sierra de les Pitas
Tacubaya
Lago de Texcoco
POPOCATEPETL
IXTACCIHUATL
S.Martin
Llanos de Apam
Tlaxcala
Cholula
MALINCHE
HUAMANTLA
Amozoc
Acajete
Nopalucan
S.Augustin del Palmer
Rinconada
PICO DE ORIZABA
S.Andres Chalchicomula
SIERRA NEGRA
Ingenio
Aculcingo
CO. PEROTE
Perote
Las Vigas
S.Miguel del Saldado
Jalapa
Rio Cumilapan
Rio Huitzuilapan
Rio Chachalacas
S.Fernando
Huatusco
CERROS CHIQUIHUITE
Amatlan
Apasapan
Paso de Ovejas
Puente Nacional
Trazo Tukott
Rio Jamapo
Soledad
Medellin

FIG. 1.

About the same prices were used on the Cordoba-Orizaba Division, 16.3 miles, *via* the line run down the sides of the Barranca de Metlac. Conglomerate was estimated at $1 and tepetate at 50 cents. Section No. 5, containing 315 435 cu. yd. of excavation and hauled embankment, was estimated at $200 120.83, and Section No. 8, which includes the Metlac Bridge, was calculated at $400 113.56. One tunnel, 700 ft. long, was estimated at $75 per lin. ft.

TABLE 3.—Work Required to be Done to Complete the the City

Division.	Distance, in miles.	Excavation, in Cubic Yards.			
		Rock.	Conglomerate.	Tepetate.	Earth.
Vera Cruz to Cordoba	67.8	137 962	....	508 632	234 442
Cordoba to Orizaba	16.3	61 695	204 809	64 571	326 119
Orizaba to Maltrata Summit	28.1	341 591	....	73 185	14 341
Total from Vera Cruz to Maltrata Summit	112.2	541 248	204 809	646 388	574 902
Maltrata Summit to Junction	67.1	122 083	....	260 732	146 976
Junction to Mexico	87.1	....	....	182 171	236 844
Total quantity to be done from Vera Cruz to Mexico.	266.4	663 331	204 809	1 089 291	958 722
Puebla Branch	29.6	51 400	....	123 181	....
Total quantity, including Puebla Branch	296.0	714 731	204 809	1 212 472	958 722

For the Road *via* Puebla.

Division.	Distance, in miles.	Rock.	Conglomerate.	Tepetate.	Earth.
Vera Cruz to Maltrata Summit	112.2	541 248	204 809	646 388	574 902
Maltrata Summit to Puebla	60.2	165 400	....	139 163	109 956
Puebla to Junction	29.6	51 400	....	123 181	....
Junction to Mexico	87.1	....	....	182 171	236 844
Vera Cruz to Mexico	289.1	758 048	204 809	1 090 903	921 702

On the Maltrata Pass-Orizaba Division, 27 miles, which includes the world-renowned "Maltrata Incline", with grades of 212 ft. (and more) per mile, the excavation price was generally $1.25, with some material at 25 and some at 45 cents. Hauled embankment ranged from 3 to 16 cents, and borrowed material was 30 cents. Masonry was $8, $12, and $13, and retaining wall was $2.50. There were three tunnels, aggregating 660 lin. ft., estimated at $75 per lin. ft. The estimated

ROADBED FOR THE LINE OF RAILWAY FROM VERA CRUZ TO OF MEXICO.

Embankment, in Cubic Yards.		Masonry, in Cubic Yards.			Retaining wall, in cubic yards.	Tunneling, in linear feet.	Iron girders.	Remarks.
From excavation.	Borrowed.	Dry.	Culvert.	Bridge.				
644 451	389 239	....	15 445	34 650	....	840	....	Two tunnels, 1-460 ft. 1-380 "
323 475	115 347	....	7 328	28 208	5 286	700	....	One tunnel.
164 432	443 374	....	....	30 021	149 094	660	40	Three tunnels, 1-190 ft. 1-110 " 1-360 "
1 132 358	947 960	....	22 773	92 879	154 380	2 200	40	
234 708	198 294	1 485	5 982	....	....	....	....	
202 182	175 869	736	4 378	....	....	....	....	
1 569 248	1 322 123	2 221	33 133	92 879	154 380	2 200	40	
129 278	320 399	945	32 423	....	....	....	....	
1 698 526	1 642 522	3 176	65 556	92 879	154 380	2 200	40	

1 132 358	947 960	....	22 773	92 879	154 380	2 200	40	
296 989	460 696	3 416	12 873	....	....	....	....	
129 278	320 399	945	32 423	....	....	....	....	
202 182	175 869	736	4 378	....	....	....	....	
1 760 807	1 904 924	5 097	72 447	92 879	154 380	2 200	40	

Bridges.	Cubic yards of masonry.	
Jamapa..............	16 000	at $12.50
Canalita..............	10 000	" 10.00
Seco..................	7 000	" 10.00
Alejo.................	1 650	" 10.00

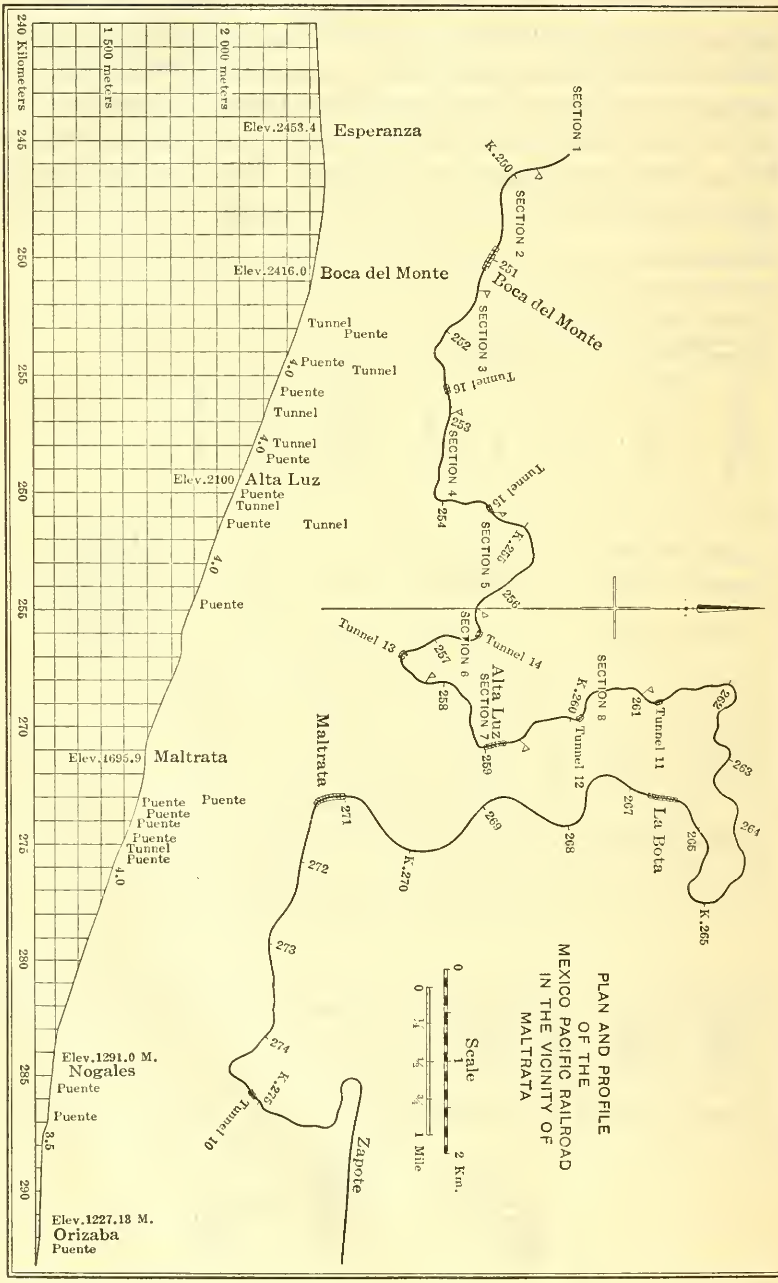
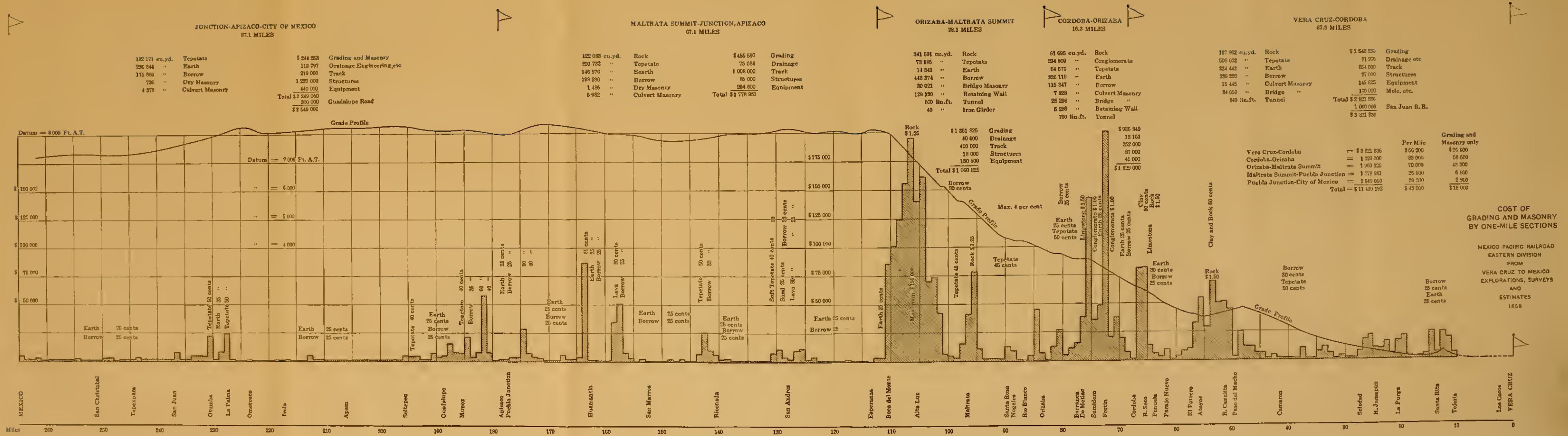

PLAN AND PROFILE OF THE MEXICO PACIFIC RAILROAD IN THE VICINITY OF MALTRATA
Scale
0 1 2 Km.
0 1 Mile
240 Kilometers
245
250
255
260
265
270
275
280
285
290
1 500 meters
2 000 meters
Elev.2453.4
Esperanza
Elev.2416.0
Boca del Monte
Elev.2100
Alta Luz
Elev.1695.9
Maltrata
Elev.1291.0 M.
Nogales
Elev.1227.18 M.
Orizaba
Tunnel
Puente
4.0
3.5
SECTION 1
SECTION 2
SECTION 3
SECTION 4
SECTION 5
SECTION 6
SECTION 7
SECTION 8
K.250
K.255
K.260
K.265
K.270
K.275
Tunnel 10
Tunnel 11
Tunnel 12
Tunnel 13
Tunnel 14
Tunnel 15
Tunnel 16
La Bota
Zapote

FIG. 2.

PLATE LXXII.
PAPERS, AM. SOC. C. E.
DECEMBER, 1915.
LOW ON
MEXICO AND PACIFIC RAILROAD.



cost of Section No. 7 was $194 832.85. A large quantity of retaining wall was estimated on this Division, 129 120 cu. yd.

The next division extends from the Maltrata Summit to the junction with the Puebla Branch near Molino San Diego, 68½ miles. The excavation and borrow prices were generally 25 cents each, although there was some excavation at 50, 61, and 80 cents, the latter being lava. There was also a small quantity of borrow at 33 cents. The estimate for Section No. 38 was only $183.85, with many less than $1 000. That for Section 51 was $86 136.02. For the first 44 miles the average cost was $3 300, and for the entire 68 miles, less than $7 000 per mile.

On the direct line from the Maltrata Summit to Puebla, 61¼ miles, the excavation prices ranged somewhat higher, 25, 30, 50, 60 cents, and $1.25, the hauled embankment, from 2 to 18 cents, and the borrowed material from 25 to 50 cents. Generally, the cost per mile was low, although it was high on two sections; on No. 9 the estimated cost was $82 642.00, and on No. 10, it was $145 615.91. The average cost per mile was $10 000.

On the Puebla Branch, 29 miles, higher unit prices were assumed, uniformly 40 cents for excavation, except on Sections Nos. 27 and 28, where 80 cents was taken for tepetate. Hauled embankment was from 2 to 13 cents, and borrowed material, 30 cents.

On the division from the junction of the Puebla Branch to Mexico, called the Western Division, 83 miles, the ruling price for excavation and borrowed material was 25 cents. The average estimated cost per mile was less than $3 000.

APPENDIX A

Extracts from Report of Mr. M. E. Lyons,* Principal Assistant Engineer.

"Shortly after the arrival of the engineer corps in Vera Cruz, they were instructed to proceed to Tejeria and encamp in the most eligible place in the neighborhood, until such time as arrangements could be made for transporting them to the interior, for, owing to the loss of the mails by the previous steamer, Mr. Escandon had not been duly informed of the departure of the engineers from the States; and as no means of transportation had been provided previous to their arrival, there was a consequent delay of nearly 3 weeks.

"This time was not wholly lost, however, as the party had to be drilled into camp life, surveying instruments had to be tested and adjusted, and the corps more perfectly organized.

"The threatened civil war, which afterward unhappily broke out, affected disastrously all the commercial interests of the Republic. The stories which reached us of the attacks made on travelers, the assaults committed, and robberies perpetrated on the highway in midday, made the party exceedingly wary, and induced us to form a semi-military organization, for our mutual protection, which we upheld ever afterward during our stay in the Republic.

"Sentries were mounted and relieved every 2 hours, during the night, and this was continued without intermission at each and every encampment, and to the vigilance exercised at all times the party no doubt owe their entire exemption from harm.

"The problem proposed for our solution is to find the most direct and practicable line for a railroad between the commercial and the political capitals of the Republic. Mr. Almazan furnishes a valuable map and barometric profiles which he had prepared of the Atlantic slopes. Between the mountains of Orizaba and Perote, the chain of the Cordilleras appears to be very high, and it is therefore deemed useless to make a reconnaissance of the depressions in the chain between these points. Northward of Perote and southward of the Peak of Orizaba the depressions seem to be deeper and more accessible. Two parties are detailed for the duty of examining these passes, and reporting on their practicability for railway purposes: Mr. Richards in charge of the party sent to examine the passes north of Perote, and Mr. Stoddart to examine those south of Orizaba.

"The route in the vicinity of the national road between Vera Cruz and Jalapa presents very unfavorable ground for a railroad. From San Juan, the western terminus of the existing railroad to Paso de Ovejas, passing through Zopilote, Tierra Colorado, and Tolome, it is hilly and unfavorable. The stream and valley of the Paso de Ovejas would be difficult and expensive. The country between Paso de Ovejas and Puente Nacional is thickly covered with underbrush of a tropical growth.

"The *barranca* at the Puente Nacional is a serious obstacle to a railroad, being about 1 500 ft. wide, and 250 ft. deep. For about 5 miles west of the Puente Nacional the ground is represented to be

* Mr. Lyons, formerly of Reading, Pa., was Capt. Talcott's assistant for many years.

favorable, and then becomes broken and unfavorable. At Plan del Rio we encounter another deep *barranca,* but not quite so wide as that of the Puente Nacional. At Rancho Cerro Gordo it is represented to be almost an impossible ground in a commercial sense for a railroad.

"A little west of this place, and near Mirador to Encerro, there exists a wide and deep valley descending to Dos Rios with a slope of some 240 ft. per mile, and to a depth of 500 ft. From this point to Jalapa is broken, but there is no unsurmountable obstacle in the way. From Jalapa to the summit of the Cordilleras at Las Vigas is a continuous mountain side, broken in places by ravines and water-courses, descending the mountain slopes and intersecting the route nearly at right angles.

"I cannot do better than give an extract from the report of the Engineer charged with the duty of exploring these mountain passes:

" 'It [the route] certainly impressed me very unfavorably from an engineering point of view. The route from Las Vigas, round by the base of Perote [Mountain] toward Jalapa, seems—as far as I could judge from a very slight examination, which was all I could give—the most favorable, all cuttings, however, will be through rock of a volcanic nature. It comes to the surface in all directions; in many cases the débris would make very fine ballast, requiring little or no breaking. Possibly a line might be had from Las Vigas by Perote Mountain to Jalapa, with a general grade of 170 ft. to the mile, but it would be both crooked and very expensive. There seemed to be a lower gap to the north of Las Vigas. I went to examine it at one point, and Mr. Stack at another. The fog came on very thick before the exploration could be completed, but, from what I could see, it appeared about the roughest country I ever saw. The ravines were from 500 to 800 ft. deep, and the sides so steep that a man could with difficulty stand on them. This lower summit is at the head of the Canada de Actopan; up the banks of this river it has been stated that a route could be obtained; it might, possibly, but the Rio de Actopan falls into the sea, considerably north of Vera Cruz, and there might be considerable difficulty in raising it out of the valley to get to Vera Cruz, except the river was practicable to the sea, which would, of course, lengthen the line considerably.'

" 'From what I afterwards saw of river routes, I do not think them well calculated for railroad lines. Even if a line could be had up the Actopan River there would be great difficulty at and near the gap in question. It would require a considerable deal of exploration to pronounce on the capabilities of the region, and time did not allow me to examine it, as it ought to have been, but my general idea is that it is the worst country I ever saw to try to get a line of railway through.'

" 'From Las Vigas the party returned to Jalapa, and spent some time in looking around there. It was supposed that a line might be had down a river that flows beneath the Puente Nacional, but the same river was crossed by the party two days afterward on the route from Jalapa to Cordoba. It is most likely impracticable to follow this river, if the whole of it be like the place where the party crossed it. It flows at the bottom of a *barranca* from 800 to 1 000 ft. deep. The sides often sheer cliffs from 100 to 300 ft. deep and upward. There

is no way out of the *barranca* except by a steep zigzag mule path. The rest of the country from this *barranca* to San Juan Coscomatepec is more or less broken and rugged. From San Juan Coscomatepec to Cordoba our path lay over more even ground, and formed almost a regular descending plane, crossed occasionally by small streams till we reached Cordoba.' "

Mr. Almazan, the Mexican engineer, who explored this section of country north of the Cofre de Perote, states his views in reference to the Jalapa route in his report, Appendix *B*, to which the reader's attention is directed.

Additional Extracts from Mr. Lyons' Report.

"Previous to making any detailed surveys, it became necessary to determine the most practicable pass in the mountains south of the Peak of Orizaba. There were three passes which might be approached from the valleys of Ingenio, Encinal, and Maltrata. The first is the Pass of San Antonio.

"*Pass of San Antonio.*—This is the nearest to the Peak of Orizaba. The summit reached the altitude of 9 200 ft. above the sea, being more than 1 000 ft. higher than the table-land beyond. This additional ascent, over what was necessary to attain to the table-land, was a fatal objection to the adoption of this pass.

"The second one examined was the next one immediately south of the San Antonio Pass and known as the Maltrata Pass.

"*Maltrata Pass.*—The altitude of this pass was found, by barometrical measurement, to be 8 110 ft. above the sea, being at the level of the table-lands, and involving, therefore, no descent to reach them, so that when the line reached this altitude the difficulties of the Cordilleras might be said to have been fully overcome.

"*Aguila Pass.*—This was the next pass examined, its altitude being nearly the same as that of Maltrata, with this difference, however, that when the difficulty of encountering this elevation was overcome, a descent into a valley west of the summit has to be made, and then an ascent to a second summit followed by a second descent to the table-lands.

"*The Aculzingo Summit.*—This summit was also examined. The summit of the first *cumbres* was lower than any of the previous passes examined, being only 7 580 ft. above the sea; but from this summit, a descent had to be made to the valley of the Puente Colorado, then an ascent to a second summit, nearly as elevated as the Maltrata Pass, then a descent to the Canada de Ystapa, followed by another ascent to the summit beyond, from which another descent was necessary to reach the valley of San Augustin de Palmar. It became very evident that the farther south our examinations extended, the more broken was the country encountered, and that we had not only one summit to overcome but several. The Maltrata Pass was the only summit examined where, the difficulties of the first ascent being overcome, no others presented themselves, and the line ran directly on the plains.

"On further examination of this Maltrata route, we exceeded our expectations by finding gradually ascending valleys, over which the road could be carried to a considerable elevation before encountering

the obstacles of a mountain line, but this portion of the line will be more minutely described hereafter.

"Having thus accomplished what was deemed the most difficult and interesting portion of our duties, our attention was turned to discovering the most feasible line from Tejeria (the point decided on for diverging from the San Juan Railroad) to the Maltrata Pass. A great obstacle stood in our way. The Chiquihuite Mountains, which lie nearly midway between the crest of the Cordillera and the sea, had to be turned either to the north or south, or entered through the gap cut by the Rio Atoyac. Mr. Almazan had previously examined the route north of the Chiquihuite Mountains, and subsequent examinations fully corroborated the views expressed by him in his report.

"A route to the south of the Chiquihuite range had also been examined. This would cross the Jamapa River at the village of that name, the Atoyac River at Cotastla, and the Obispo River at Paso Obispo. This route was favorable, with the exception of the portion of the line between the eastern bank of the Atoyac and the Village of Los Negritos, where the ground was extremely broken and unfavorable. Another route was suggested; this would diverge from the San Juan Railroad at a point 4 miles from Vera Cruz, and, crossing the river a short distance below the confluence of the Atoyac and Jamapa, thence occupying the divide between the Rio Blanco and Rio Obispo, the line might be run to a point opposite Cordoba, with very little bridging or culvert masonry. This route would involve a considerable deflection to the south, and, therefore, might increase the length of the line from Vera Cruz to Cordoba by at least 10 miles.

"It having been thus determined: First, that the Maltrata Pass, was the best one for the railroad for many miles south of the Peak of Orizaba; second, that the line must pass through the defile of Ingenio; and third, that a line passing through the gap where the River Atoyac breaks through the Chiquihuite Mountains would offer the shortest practicable line between Vera Cruz and Ingenio.

"The party received instructions early in April, 1858, to adopt that portion of the San Juan Railroad lying between Vera Cruz and Tejeria, and, from this initial point, run a line over the most favorable ground to the Atoyac Gap in the Chiquihuite Mountains, thence to Cordoba, and through the Ingenio Valley to the Maltrata Summit.

"Previous, however, to the location of this line, an experimental line had been run from Camaron to San Juan de la Punta, and also reconnaissance made of the country in a straight line between La Soledad and San Juan de la Punta, all of which went to prove that the country between the points referred to was very much broken and nearly impracticable for railway purposes.

"The line located in obedience to the above instructions diverges from the San Juan Railroad ½ mile west of the Tejeria station on that road, and runs thence in a southwesterly direction and in a very direct line, passing a little to the north of Rancho Nuevo, Mata Cordera, La Purga, Arroyo de Piedra, Mata Judio, to the considerable village of La Soledad, where it crosses the River Jamapa by a bridge, 102 ft. high and 150 ft. long. The first 6 miles of this line crosses low sand hills and valleys, which, although covered at that time with verdure, were once the shifting sands blown up from the

Gulf of Mexico. At the end of these 6 miles, the line enters on ground of a new and peculiar formation, called, in Mexico, 'tepetate', a name applied to the drift or deposit of detritus, the débris of the rock which forms the materials of the Cordilleras. This detritus extends to an immense depth. In the first 10 or 12 ft. from the surface there are water-worn stones and sand cemented together; below this, the material is composed of gravel, then sand, and, lower still, sand of a very fine quality. This tepetate rock is soft when first removed from its bed, but when exposed to the sun for a few days becomes harder, and might be used as a building stone in places where it would not be subjected to great pressure. When first quarried it is easily pulverized, and makes an excellent constituent for mortar or cement.

"The line from Tejeria to La Soledad, as before described, is undulating, the heaviest grade at any place being 60 ft. per mile. The line after crossing the Jamapa River at La Soledad occupies the south bank of that stream. Thus availing ourselves of a most remarkable natural feature in this section of country, which is that the southern bank of the Jamapa River is the ridge of the water-shed from which the streams fall into the Rio Atoyac, so that by running the line as near the bank of the river as practicable, and occupying the plain at the point where it began to resolve itself into ridge and valley, a way was found for the roadbed already prepared by natural means. The line, therefore, runs from the crossing of the Jamapa for a distance of 17 miles near the bank of that river, the work of graduation and masonry on that portion of the line being exceedingly light; the grades, however, are heavy, rising at first 70 ft. per mile, then 80, 85, 90, and finally running up to 110 ft. per mile. The line has here reached a point 36 miles from Tejeria, where the Jamapa runs no longer in our true direction, but takes a northwest course to Pueblo Viejo. The line at this point leaves the bank of the river, and runs southwest, crossing a stream at Paso Tablo and encountering near the Rancho Santa Teresa, the Rio de San Alejo, an inconsiderable stream at this point, running in a *barranca* of no great depth. From the crossing of the San Alejo, the line descends to the Canalita, a small stream which takes its rise near the Hacienda Defensa and runs along the eastern base of the Chiquihuite range. The line crosses the Canalita, at an elevation of 90 ft. and thence it occupies the slope of the Chiquihuite Mountains to the Gap of the Atoyac. From the crossing of the Jamapa River at La Soledad to the crossing of the San Alejo at Santa Teresa the line is comparatively open and free from woodland, if we except a few groves of trees which here and there dot the plain; not so, however, the ground between the Rio San Alejo and the Chiquihuite Mountains; this country is thickly wooded, with underbrush below, and parasitical plants and creeping vines hanging to the branches of the trees above, in places almost excluding the light and rendering it a matter of extreme difficulty to discover the best route, and to cut a way through it. The Chiquihuite Mountains are densely wooded. Many of these woods will be available for the railroad. The live oak which here abounds will form a lasting sill, and fuel for a century to come can be obtained on these mountains.

"The line, as above stated, occupies the eastern slope of the Chiquihuites, from the crossing of the Canalita to the gap made by

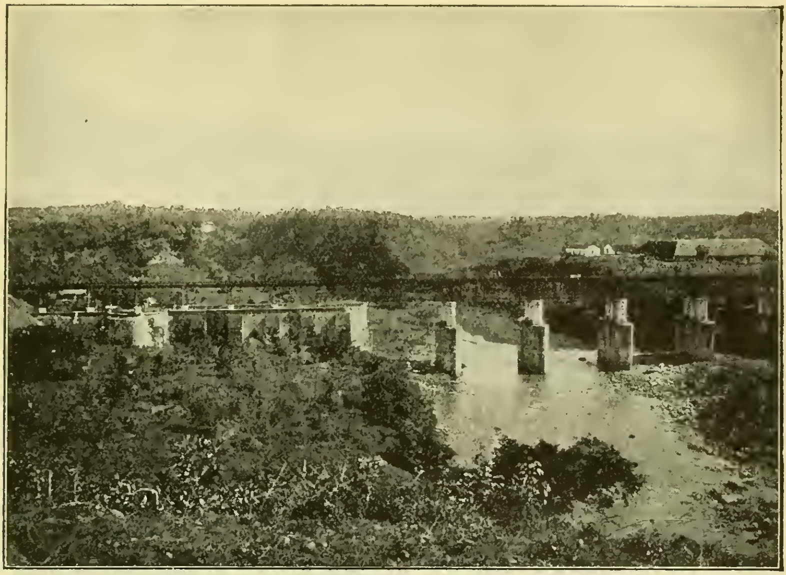


FIG. 3.—SOLEDAD BRIDGE, OVER JAMAPA RIVER, 42 KM. (26 MILES) WEST OF VERA CRUZ. ORIGINALLY A HIGHWAY BRIDGE COSTING $328 109. OVER THIS BRIDGE MR. M. E. LYONS BUILT A WOODEN STRUCTURE COSTING $200 000, WHICH LASTED UNTIL 1868, WHEN IT WAS REPLACED BY AN IRON BRIDGE.

FIG. 4.—SAN ALEJO BRIDGE, OVER THE RIO DE SAN ALEJO, 82 KM. (51 MILES) WEST OF VERA CRUZ. 100 METERS (328 FT.) LONG.

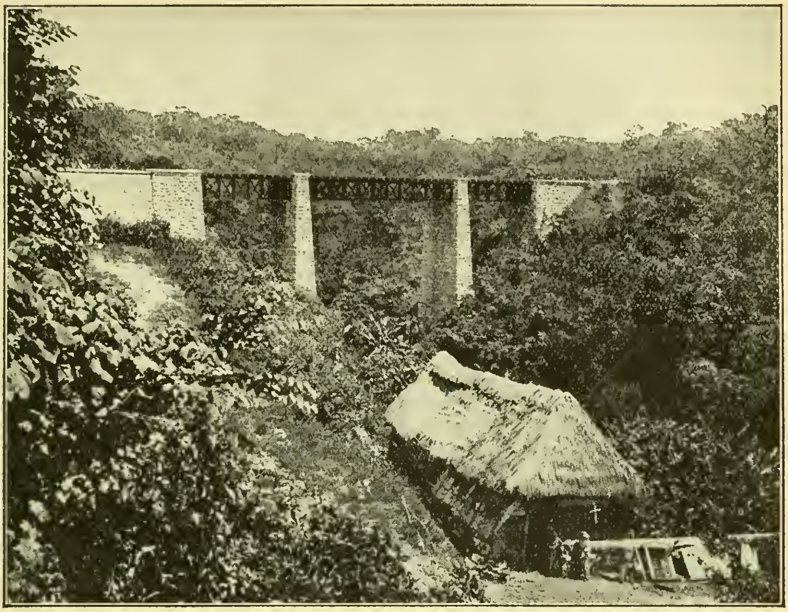


FIG. 5.—CHIQUIHUITE BRIDGE, OVER RIO CHIQUIHUITE, 83 KM. (51½ MILES) WEST OF VERA CRUZ. 67 METERS (220 FT.) LONG.

FIG. 6.—ATOYAC BRIDGE, OVER RIO ATOYAC, 86 KM. (53⅓ MILES) WEST OF VERA CRUZ. 100 METERS (328 FT.) LONG.

the Atoyac in this mountain range. About 1½ miles below the crossing of the Canalita there is a vast accession to its waters by the Rio Chiquihuite which gushes full size out of the mountain side. This source of the Chiquihuite is about ¼ mile above where the river is bridged by the public road. At the gap in the Chiquihuite, the line crosses from the north to the south bank of the Atoyac River, at an elevation of 304 ft. above the water. It will be possible hereafter to lessen the height of this bridge by increasing the grade—now nearly level—on both sides of the Atoyac River, and to introduce such other modifications of the line as future explorations may prove advisable. About a mile west of the crossing of the Atoyac the line leaves the mountain side and enters the Valley of El Potrero; then, occupying the divide between the waters of the Atoyac and the Seco, it passes through a densely wooded country with light earthwork, and but little masonry, till it reaches the Rio Seco, a short distance west of the Village of Peñuela. The ground rose so rapidly in the Valley of El Potrero, although but little indication of this fact would be given to the traveler with his unassisted eye, that grades of 132 ft. per mile became necessary. The line crosses the Rio Seco 60 ft. above the water, and then, curving to the south, intersects the public road a short distance west of Peñuela, and passing over some broken ground to San Miguelite, from which it runs to a point opposite the City of Cordoba, occupying a narrow ridge which offers both favorable ground and alignment. The grade from San Miguelite to Cordoba is 132 ft. per mile.

"An attempt was made to carry the road in a direct course from the point opposite Cordoba to Las Animas, but it was found that a grade steeper than 132 ft. to the mile would be required, and therefore it was deemed advisable, rather than increase this grade, to make a slight detour for the purpose of lengthening the line by way of Dos Caminos to Animas, and thence to the crossing of the Barranca de Metlac. It is probable that on final location this line would not cross the stream near San Miguelite, but would occupy the ridge nearer to the public road, and run closer to the City of Cordoba than the present line, which runs nearly a mile south of that city.

"The country is tolerably open from the crossing of the Rio Seco to the City of Cordoba.

"This city has a population of about 5 000 to 6 000. From its general appearance it seems to have had more prosperous days. It contains a very fine parochial church, several religious institutions, a barracks, some very good stores, and hotels. The soil in the vicinity seems to be very rich, but it is not much cultivated except in the immediate vicinity of the city, where vegetables, bananas, oranges, and coffee are raised. The plains which surround the city for many a square league are used for pasturage.

"The line has been described to the point where it crosses the Barranca de Metlac. This *barranca* is the most formidable obstacle to be encountered at any point between Vera Cruz and Orizaba. It is a deep gorge cut beneath the level of the plain. From the grade points on each bank, the distance is 1 085 ft. across, and the depth, from the grade to the level of the water in the bottom, is 346 ft.

"The subject of bridging this *barranca* requires considerable study,

and the preparation of plans and details in order that the subject may be viewed in all its bearings, and the question of economy and stability may be fully discussed.

"The erection of such a bridge will necessarily occupy much time, and therefore examinations were made, to ascertain if it were practicable to obviate the building of this high bridge, or to ascertain if some temporary expedient could not be resorted to, by which the road could be worked, before the high bridge was completed. A line with a descending grade, was run down the eastern slope of the *barranca,* and advantage taken of a point where a bend occurs in the stream, and where the valley widens, to cross the river at that point, and curving to the south on a radius of 400 ft., to the opposite bank of the *barranca,* ascending thence to the plain. This line would cross the river with a bridge 500 ft. long, and 140 ft. high, but it would add 2 miles to the length of the road.

"On gaining the western slope of the *barranca,* the line occupies a favorable valley, and, with the exception of crossing three or four streams, meets with no important obstruction until it reaches Cocolapan, and the City of Orizaba. From the Barranca de Metlac to Cocolapan the grades do not exceed 106 ft. to the mile.

"From the City of Orizaba westward the line runs through valleys bounded by majestic mountains, the presence of which greatly disturbs preconceived ideas of distance, for wide valleys are dwarfed in their vicinity to the appearance of one-third their actual width.

"The line occupies nearly the center of the Ingenio Valley, and west of the Town of Ingenio it curves to the north, and runs through the Valley of Encinal, passing a little to the west of the *hacienda* of that name. This valley rises so abruptly that it becomes necessary to introduce grades of 200 ft. to the mile; near this the head of the line curves to the south and enters on the slope of the *barranca* which leads from the Valley of Maltrata into that of the Encinal, and after encountering some heavy work, in rounding one or two bluffs, it emerges from this narrow gorge into the Valley of Maltrata at the north end of which stands the Indian village of the same name, containing a population of several hundred.

"The line of survey touches the north end of the village, where a light grade is introduced to enable the company to make a stopping place, and also to give the engine an opportunity to accumulate steam to propel the train up the steep grades which intervene between Maltrata Village and Maltrata Pass. From Maltrata the line ascends, with a grade of 203 ft. to the mile, for 3 miles, where another piece of light grade is introduced for the purpose before mentioned. The line has now reached an altitude of 6 100 ft. above the level of the sea; the valleys reach no higher, and the remainder of the ascent must be made on the steep and rugged sides of the mountain. Fortunately, there is only 8 miles of mountain line, a result certainly not looked for when we first viewed these formidable barriers from Tejeria.

"The ascent from the head of the Maltrata Valley to the Maltrata Summit is made with grades averaging 200 ft. to the mile, increased on straight lines to 212 ft., and proportionately reduced on curves.

"From the Maltrata Pass to the City of Mexico the railway can be carried over a series of plains, some of them separated one from an-

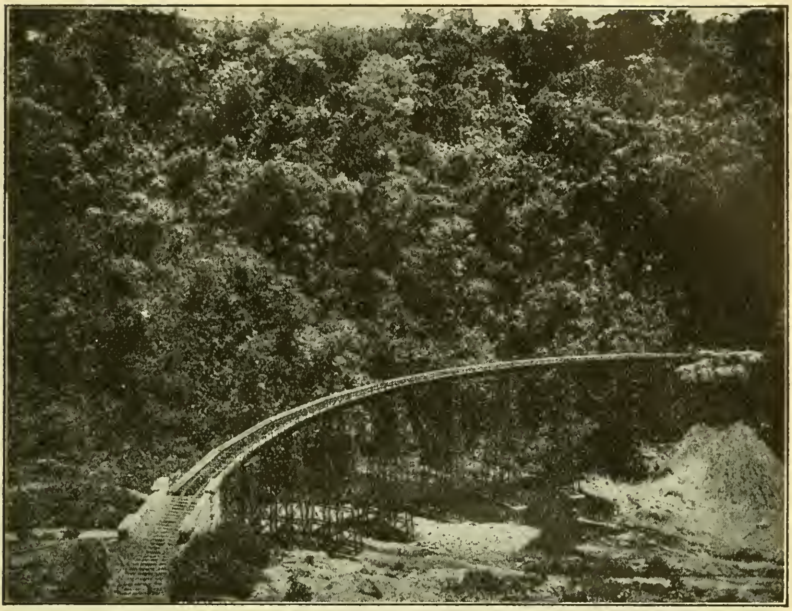


FIG. 7.—METLAC BRIDGE, OVER BARRANCA DE METLAC, BETWEEN CORDOBA AND ORIZABA, NEAR FORTIN, 116 KM. (72 MILES) WEST OF VERA CRUZ. NINE SPANS OF 50 FT. EACH. 80 FT. HIGH.

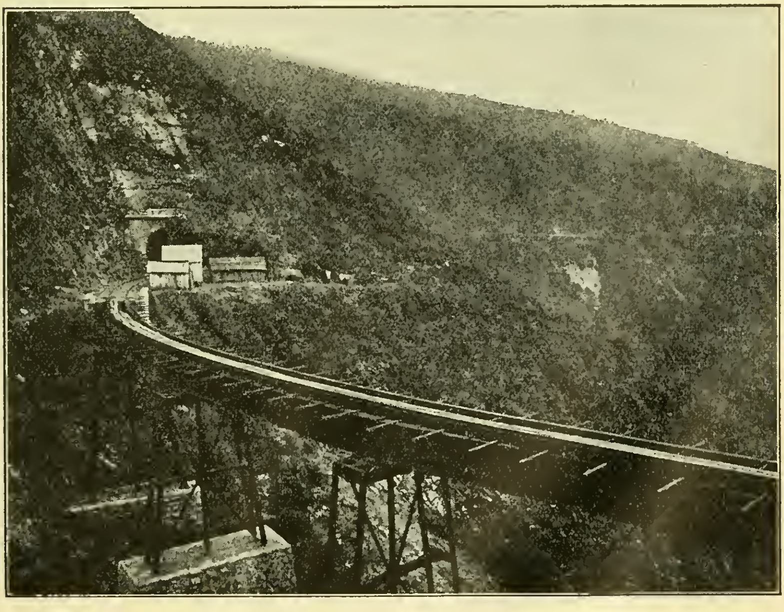


FIG. 8.—WIMMER BRIDGE, ON THE MALTRATA "INCLINE", 170 KM. (105¾ MILES) WEST OF VERA CRUZ. FIVE SPANS OF 17 M. (55¾ FT.) EACH. "HORSE SHOE BEND" IN RIGHT DISTANCE.

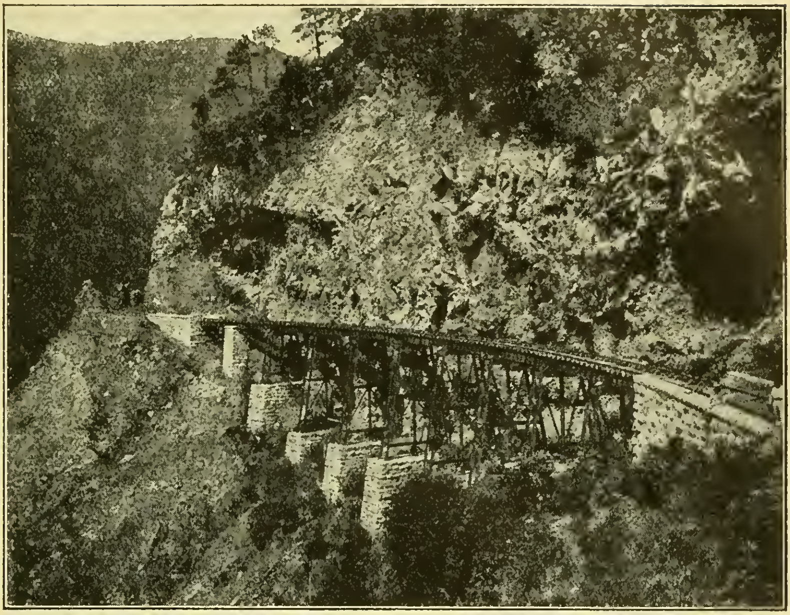


FIG. 9.—INFIERNILLO VIADUCT, EAST OF MALTRATA, 150 KM. (93 MILES) WEST OF VERA CRUZ.

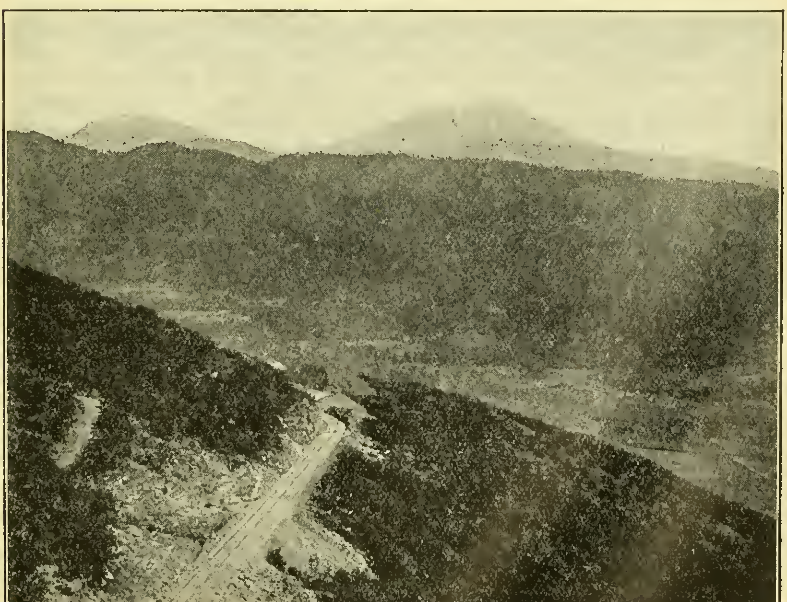


FIG. 10.—PANORAMIC VIEW, HEIGHTS OF MALTRATA. VOLCANO OF ORIZABA IN THE DISTANCE.

other by low trap dikes, others, again, are regular steppes, one plain being at an elevation of from 200 to 300 ft. above the other, but the ascent or descent can always be made by running along the slopes of the mountains. The direct line between the Maltrata Pass and the City of Mexico would leave the important City of Puebla nearly 30 miles to the south, rendering necessary a branch line of that length to connect it with the main line.

"To obviate this, explorations were set on foot to ascertain the practicability of connecting the valleys of Puebla and Mexico, by a tolerably direct line. It was soon discovered that no practicable route for a railway lay in the vicinity of the public road near Rio Frio, which at its greatest elevation attains an altitude of 10 500 ft., or nearly 2 miles in vertical height above the level of the ocean. Explorations were also conducted in the vicinity of San Martin to ascertain the practicability of finding a route for a railroad between that point and the Hacienda San Nicholas. This line would involve the building of 14 miles of mountain line, with a uniform grade of 106 ft. per mile. It was found, moreover, that the saving in distance would not be very great, as considerable northing would have to be made, in order to turn the chain of high mountains near Apam. A line running along the base of the western slope of Mt. Malinche, and near the banks of the Rio Atoyac, presented the best line for connecting by railroad the City of Puebla with the plains of Apam. The surveyed line, therefore, was run with the view of making the City of Puebla a point on the main line of railroad. The line descended with the grade of the plain from the summit of Maltrata to Cuesta Blanca, a distance of 8 miles, over ground that required but little preparation for the roadbed. At Cuesta Blanca the line enters on the slopes of the mountains which separate the plains of Esperanza from the plains of San Augustin de Palmar, and descends with a grade of 86 ft. per mile to the latter plain, and, skirting for a distance of several miles the northern edge of the plain, it leaves the Town of San Augustin de Palmar 2 miles to the south, the Town of Quecholas about a mile to the north, and runs close to the Towns of Acacingo and Amozoc. The mountain portion of this line, some 5 miles of it, is really the only objectionable portion, for, after reaching the plains of Palmar and Acacingo, it runs with most excellent alignment and light grades to the City of Puebla. The whole of this valley is thickly populated and well cultivated, and is interspersed with towns and fine *haciendas*. The phenomenon of whirlwinds and the mirage were made familiar to us here. At several points in the valley the whirlwinds may be seen lifting the light grains of sand and forming columns 60 ft. in height which are carried in every direction by the currents of air. Some of the gentlemen were much deceived in the appearance of water. The light reflected at a favorable angle from the sand gave portions of the plain the appearance of a sheet of water, a cloud would at times intervene and the phantom lake would instantly disappear.

"The surveyed line did not enter the City of Puebla, but ran a little north of the Guadalupe hill, and a short line was afterwards surveyed to connect the main line of survey with the plaza in that city. The line of survey runs thence along the base of the mountain of Malinche (the base of which extends over a diameter of 20 miles)

and intersects a great number of *barrancas,* the floods from this elevated peak having cut deeply into the slopes of the mountain. These *barrancas,* in many cases, were as deep as they were wide, and in some places they appeared to be so narrow, that it seemed possible to jump from one side to the other, and yet they were very deep. The upper portions consist of a multitude of channels spreading out with fan-like appearance. These *barrancas* obtain their greatest depth in a few hundred yards. In the rainy season the waters run with great velocity until they are somewhat checked by the flattening of the mountain slopes; they then begin to deposit the material they held in suspension, and from that point to the banks of the Atoyac, at the base of the mountains, these *barrancas* become shallower.

"The strata of the lower portion of the mountain are hidden from view by a thick deposit of detritus, which forms the surface and constitutes the material of this and all the other mountain slopes in the neighborhood of the line of survey between Puebla and the City of Mexico. The line, after running for nearly 30 miles due north from Puebla, enters the plains of Apam, and, running a short distance south of the town and lake of that name, proceeds for many miles over nearly level plains, with light grades, and still lighter earthwork, until it reaches the point where it begins to ascend to the Otumba summit. The elevation of this summit is 8 272 ft. above the sea, and, therefore, is the highest ground over which the line passes between Vera Cruz and the Capital. The excavation through the Otumba summit will be through 'tepetate'. From the Otumba summit the line descends into the far-famed Valley of Mexico, passing near the Town of Teotihuacan, Tepexpam and a number of villages and *haciendas,* and, running for 15 miles along the shores of the salt-water lake, Texcoco, joins the Guadalupe Railroad about 3½ miles from the City of Mexico and ½ mile from the northern terminus of this little road at Guadalupe.

"The surveyed line from Puebla to Mexico describes nearly the periphery of a semicircle the center of which would be in the neighborhood of Popocatepetl. It is 112½ miles from Puebla to the junction with the Guadalupe road, and 116 miles between the two cities. The grade descending from the plains of Otumba to the Valley of Mexico is 90 ft. to the mile, but this can be reduced, and also the grades at other points, by lengthening the line where these heavy grades occur. The line passes near the Towns of San Pablo, and Santa Ana, and some 2 miles east of the ancient City of Tlaxcala. The land here produces all cereals and fruit of the temperate zone, and the plains of Apam produce the finest specimens of the aloe, or maguey plant which supplies the Cities of Puebla, Mexico, and Toluca with pulque, in which article there is an immense trade.

"The great advantage, says Baron von Humboldt, of the maguey growing on plains elevated from 7 000 to 10 000 ft. above the sea, is that it is not affected by change of climate, and that its firm vigorous leaves withstand hail, frost, and drought. A vigorous maguey plant will produce 1½ gal. of pulque per day for a period of 5 months. The Indian calculates the produce of the maguey at 150 bottles, and the value of a plant near its efflorescence at from $4 or $5 to $8. The maguey is not only the vine of the Aztec, but its fibrous leaves sup-

plied him with paper on which he painted his hieroglyphics, its prickles which terminate its leaves served for pins and nails, and were used by the heathen priests for piercing their arms in their acts of expiation. They also obtained thread from the maguey plant, and its pulpy substance, when roasted, formed the sustenance of a large portion of the population.

"The City of Mexico, containing a population estimated at from 170 000 to 200 000, stands near the center of the Valley of Mexico. The valley is of elliptical form, the greatest diameter being 48 miles, and its least diameter 32 miles. One-twentieth of its entire surface is covered with water. The whole valley, except to the north, where the railway enters it, is shut in by high porphyritic mountains.

"On mature deliberation, it was concluded to run another line from the Maltrata Pass, by way of Nopalucan and Huamantla, to join the line run by the way of Puebla on the Plains of Apam.

"This section of country was favorable for a railroad route, and if it should be finally adopted for the main trunk line, giving Puebla a branch line, it will save 19 miles in distance to travelers between Mexico and Vera Cruz over the line passing by Puebla. It will not add anything to the distance to be traveled between Puebla and Mexico, but will add 39 miles to the journey to be performed between Puebla and Vera Cruz.

"Detailed estimates of the cost of these lines, and tables of grades will be submitted in the latter portion of this report."

APPENDIX B

Translation of Report of Mr. Pascual Almazan.*

"The geodetic positions of the points from which observations have been made, and of others which have been sighted upon from these, also the altitudes determined by barometric leveling, are already included in the map and profiles presented to Mr. Talcott, and for this reason I will not speak of them, except to remark briefly as to the position of Vera Cruz.

"This had been ascertained at first by its longitude and latitude as computed by Baron von Humboldt, but having computed the positions of La Soledad, Arroyo de Piedra, Santa Ana, Matacordero and Rancho Nuevo by the triangles formed with the Orizaba Peak and El Cofre de Perote, respectively, and having combined the positions so obtained with the data of partial distances and directions, the result was that, to make each of these agree with Vera Cruz, it was necessary to apply to this port a correction of nearly 1′ 4″ in arc with respect to its longitude calculated from Mexico City.

"Undoubtedly, the position of Vera Cruz is more correctly computed than that of the capital, as it is based on many data presented and discussed by Oltmans in the astronomical observations of Baron von Humboldt; therefore, the final correction with respect to the Paris or Greenwich meridian should be made on the longitude of Mexico, although in the maps in which the starting point is the meridian of this capital, as is the one made for the Escandon Railroad, the modification would be made on Vera Cruz.

"The electro-magnetic telegraph being now established between both cities, it is easy to determine by it, with all the accuracy of which the determination of local time is capable, the distance or difference in longitude between these two cities and in the same way that of several intermediate points, so as to have verification data.

"Passing now to the main object, I must say that several of the reconnaissances made have only shown a negative result, that is, it has been found to be impossible to carry the railroad through the reconnoitered places. The time spent in these reconnaissances, nevertheless, may be deemed as not wasted, because it saves the time that (had these not been made) would have been spent in taking the topography and levels of the lines, which may now be deemed as impracticable.

"In this category is that of Pueblo Viejo and Ixhuatlan south of the River Jamapa, because, the direct pass from Ixhuatlan to Orizaba not being possible, it would be necessary to go up to Coscomatepec and come down immediately to the last named city [meaning Orizaba, probably] some 400 meters. This supposes:

"1st. The necessity of passing through Orizaba (if this line is adopted), which is absurd, as the only available pass for a railroad is the Gargante del Ingenio; and

"2d. That it is possible to build a grade for the road in the stretch between Pueblo Viejo and Ocotitlan, which is really impracticable if

* A Mexican engineer who made some reconnaissances of the Jalapa route previous to the arrival of Capt. Talcott in Mexico.

we consider the narrowness of the pass, its sinuosities in both horizontal and vertical directions, and the impossibility of widening at a reasonable cost a road made in the most broken part of Sierra del Chiquihuite, an upheaved limestone formation with the probable opening toward the north, and which, therefore, presents at this place a fractured edge.

"If the route through Orizaba is selected two lines are presented for reaching this city: One which almost coincides with the present road, and another passing south of San Juan de la Punta, which may pass through Cordoba or Zapacpita. If the first is taken, because special study indicates the possibility of a passage through the hills between La Peñuela and Olmeacar, there will be the advantage of a cheap crossing of the Metlac River, but only at the expense of attaining the height represented by the Tuzpango slope. This will certainly involve heavy work, but it is probably best to cross at this point.

"It may be possible, if the Chicahuastle foundation is not too brittle to overcome this height as indicated in the attached figure, by giving to the route the maximum slope of the Semmering.

"To cross the Metlac River further up than El Fortin, it is necessary to choose the route of La Peñuela and Cordoba, and the selection would probably depend on the comparison of costs respectively computed for the two routes, but the fact that passing through Zaponpita will save more than a league (4 240 meters) in the distance from Peñuela to Orizaba, should be taken into consideration.

"The selection of another line that would coincide frequently with the National road and would go through La Peñuela, or perhaps through the south of El Potrero and Amatlan, might be directed to Zaponpita or Zacatepec in the above named manner, depends on the detailed inspection of the gorge of Atoyac between El Chiquihuite and El Potrero.

"It may be possible, by turning to the north of Tres Encina and Paso del Macho, to make with relative ease the crossings of the small rivers of San Alajo and Chiquihuite, and passing along the west bank of this, follow parallel to its course in a south-southeast direction until reaching the gorge, thus passing to the eastern bank of the Atoyac, crossing it at the easiest place, or taking the west bank and entering the Potrero.

"It is truly difficult to execute these plans, and I only indicate them with the greatest reserve, because I wish to show that the railway would be shortened by about 4 leagues, which would be needed if it was attempted to go around the side.

"In this way the Jamapa would be crossed a short distance to the west of Medellin and taking the south bank of the river above mentioned, the point to the north of Tres Encinas would be met.

"Having passed the gorge of the Ingenio (for which no other pass can be substituted), we have at the best to ascend the Maltrata heights and those of Aculcingo; the first presents the advantage of the least distance to Morelos and the difficulty of passing from El Encinal to Maltrata through a deep and long gorge; the second invites because of its easy access to Aculcingo, at the foot of the mountain chain, and because in its vicinity (as may be seen on the map) is the Royas Creek over which a bridge of least altitude may be built, especially if a small tunnel is opened, thus further diminishing the altitude.

"If this route is selected, it would probably continue through Chapulco, to Carmen or to El Camero and other points that will be shown on the small-scale map that is being prepared.

"The line from San Antonio de Arriba where the barometer indicates 511 mm. is, of course, impracticable.*

"The river of Santa Catalina which belongs to the western slope of the mountain ranges and begins about 3 leagues south of the *hacienda* of San Diego would make the route of considerable length, and has at its highest point about the same altitude as that of Aculcingo, and, after ascending it, it would be necessary to descend about 500 meters to the south, only to go up the same distance by the Tehuacan route.

"It seems, therefore, that this line should not be examined.

"Between the bed of the Jamapa River and the National Bridge there are no favorable routes by which to ascend to the central plateau, and, for this reason, the examination of this region could be useful only if an easy access was found to reach a considerable height, and if, furthermore, it were easy to pass from this point to the route to be selected in the direction of Orizaba or Jalapa; but this second condition cannot be considered, because, whether running north or south, it is necessary to cross the many rivers and ravines of great depth that descend from the Cofre and Orizaba peaks, respectively.

"There remains, therefore, only the line through Jalapa, which on the National Road presents the considerable ravines of Paso de Obejas, Puente Nacional, Plan del Rio and Dos Rios, the difficult roundabout route of Cerro Gordo, and an extremely broken section from San Juan and El Zopilote to Tolome. I believe this line also should not occupy the attention of the engineers.

"This does not apply to the Octapan River which can be reached by describing a quadrant from the west of Antigua up to the Octapan Bridge, crossing there the river of this name to take advantage of the plain of this same town, recrossing the river in front of Saetal and continuing to the west on the course of the Abas or on the adjoining argillaceous formations until reaching the height of Las Bigas.

"This line presents the advantages of a continuous ascent without the necessity, in any case, of losing any of the altitude gained; the advantage of being on the route that most directly, and therefore, with the least length of line, leads toward Mexico; and last, the advantage of crossing the State of Vera Cruz on lands of small value.

"The disadvantages of this route are that it deprives the railroad of the commercial branch to Oajaca and of frequently having to form the roadbed on lava formation where cuts are difficult and would constantly require filling, thus losing the advantages to be had in railroad construction when a line can be run where the cuts are equal to the fills.

"There is also another feature that might endanger the stability of the road. It seems that the lava formation is submarine, that is, the outside was suddenly chilled and when the interior of the current diminished in volume, many geodes were formed, frequently very near the surface, which in places have been uncovered by the mechanical breaking of their light shells and in places form cavities of such

* The altitude was not computed because, the barometer being out of order, its indications were not reliable.

capacity that they absorb all the waters that flow from the slopes of this long valley. It is true that these disadvantages may be avoided by running over the argillaceous deposits at the sides, but even thus it will be necessary to cross the unstable strata for lengths totaling perhaps 6 000 meters.

"I believe the preceding indications are sufficient to enable the chief engineer to know what reconnaissances should be made in order to obtain the necessary data for deciding on and laying out the route.

"I will only add that, having had the opportunity to compare frequently the oscillations of the barometer in Vera Cruz with the simultaneous ones of another instrument in a different place, I have observed that the meteorological disturbances were not equally recorded by both instruments.

"The sinuosities and configuration of the country, the relation of different places in the valleys of the streams, make unnoticeable the small variations that may be seen in their full intensity at a place as open as Vera Cruz. This circumstance has given me, for the same place, altitudes that vary by more than 30 meters, and for this reason I believe it is necessary, hereafter, to try to diminish this cause of error.

"It may be advisable to place barometers for comparison in three different places, so as to compare the observations made in three different zones. The one at Vera Cruz would be used as a reference for the observations from the coast to the line from Paso del Toro in the valley of Adapan to San Juan de la Punta and Trapiche de Mesa. That at Orizaba would be used from this line to the crest of the range, and that of Mexico (or perhaps that of Puebla would be best, as it is a central point of the region) would provide the data for the computations of the central plateau.

"In this way the barometer, which is a meteorological rather than a mathematical instrument, would probably approach the former more nearly, because the non-computable variations which are due to atmospheric phenomena would be in part eliminated.

"I close this brief description of the local data because the scientific facts are all available to the Chief Engineer.

"Orizaba, January 9, 1858." "P. Almazan."

APPENDIX C

Letters from Mr. William R. Eastman,* Transitman on the Original Surveyed Line for the Maltrata Incline.

"I have read with great interest the report of Capt. Andrew Talcott regarding the location of the Mexico and Pacific Railroad in 1858. I was one of the engineering party brought by Capt. Talcott from the United States, and landed with them at Vera Cruz on January 4th of that year. I was in charge of transit work for one of the field parties, and, in that capacity, ran the experimental line around the Maltrata Valley in July following. In searching for a way from the City of Orizaba to the high table-land, involving a rise of 4 000 ft., within a distance of about 16 miles in a direct line, the ascent at Maltrata was not the first to be tried. At that point there was no wagon road, but, instead, there was an excellent path for horses and pack trains reaching the summit by numerous zigzags. All wheeled vehicles, however, including the daily stages to and from the Capital, were obliged to climb the mountain by the *"camino real"*, the broad national highway which led up through Aculcingo, some 20 miles to the south. As this route had been chosen for the wagon road, the obvious presumption was in its favor, and the pass at Aculcingo was first examined.

"This was done in February with great thoroughness by two field parties setting out from a common point on a level shelf more than half way up the slope and working thence in opposite directions, one up and the other down. Neither party succeeded in making a satisfactory connection, either with the summit or with the valley below. The result was also disappointing with regard to distance and probable cost of construction, and the attempt to locate at this point was definitely abandoned.

"Before the mountain problem was again taken up, in the following July, there had been a diligent search for a better route, and it was found that the pass at Maltrata offered certain positive advantages.

"In going by Aculcingo the stage road made a considerable detour to the south. This elbow would be cut off and the distances to Mexico materially shortened by going up at Maltrata. Besides, there was, on the north side of the Maltrata Valley, a mountain wall in the form of a buttress or spur jutting out squarely to the east from the summit of the pass and, at a distance of 5 or 6 miles, turning north, bounding from that point a transverse valley parallel with the main crest of the table-land.

"The sides of this great mountain spur, lying directly along the general course which we desired to follow, afforded abundant space for prolonging or shortening a railroad line so as to fit the particular grade which might be taken as a maximum. Capt. Talcott, in his report, states very clearly his views respecting a maximum grade, and the topography of Maltrata suited his purpose.

"My party began at the summit at Boca del Monte to cut a line to the valley, which was in full view about 2 000 ft. below. We could make no use whatever of the steep zigzag trail. My instructions were

* Now living in Albany, N. Y.

to set the transit at a depression of about 4 ft. in the hundred and to follow the mountain side wherever the line might carry me. Progress was slow. Three or four men were required to cut an opening through the dense growth of small trees, bushes, and vines, so as to permit a sight from one angle to the next and to allow measurements with the chain. A leveler accompanied the party to note and correct the grade.

"Twice it was necessary to triangulate across abrupt ravines. After some 3 weeks of difficult work we came out on the floor of the valley with field notes for about 11 miles of line, substantially the same, at least in general outline, as that on which the road was built, and which is still shown by the railroad in a printed diagram on the time-tables of the company.

"When the maximum grade for such an incline was determined there could scarcely have been a better location for the railroad ascent to the table-land.

"William R. Eastman."

"Albany, N. Y., March 6th, 1915."

The Survey of a Route for The Mexican Railway.
By William R. Eastman.

"The Mexican Railway, connecting the City of Mexico with Vera Cruz, was opened for public use on January 1st, 1873. The completion of this line between the capital and the nearest and most important seaport was recognized as an event of national importance, and the President of the Republic, Lerdo de Tejada, led the public rejoicings. The road was 264 miles long, had been built at great cost, and represented the planning, working, and vexatious delays of at least 20 years. M. Romero* places the extent of time required at 36 years. From him we learn that a concession for a railroad to Vera Cruz was granted in 1837 and another in 1842. Notwithstanding large appropriations made by the Government, only 11½ miles were built up to November, 1850. At the time of the United States' invasion, in 1847, there were no railroads.

"*The First Railroad.*—The first 15 miles were built from Vera Cruz through Tejeria to the small village of San Juan. In 1858, cars on this road were drawn by a locomotive made in Belgium. In 1854, a short experimental railway, a sort of miniature model of a railroad, was set up in the Valley of Mexico, running out about 3 miles to the suburb of Guadalupe, where was a famous shrine of the Virgin to which frequent pilgrimages were made by all classes of people. This road was wholly of American construction, the rails and other equipment having been hauled 260 miles from the coast. The locomotive was built in the shops at Paterson, N. J., and brought up in detached pieces which were put together by American mechanics. This little road was convenient and popular.

"In 1857, a new concession for a railroad to the coast was obtained from the Mexican Congress by Don Antonio Escandon, a wealthy owner of mines and factories, who paid $1 000 000 into the national treasury and at once entered upon the undertaking with an earnestness worthy of his purpose.

* See his article in the *International Review* of November, 1882, Vol. 13, p. 480.

"*A Mountain to be Climbed.*—The route was evidently difficult, because there was a mountain which must be climbed. The broad table-land of that part of Mexico is, in round numbers, 8 000 ft. higher than sea level. This great elevation is reached in the first 100 miles from the coast. If the slope of the land were uniform and unbroken, an average rise of 80 ft. for each mile, while troublesome, might not be thought a serious obstacle; but the ascent is neither even nor uninterrupted. Starting off with an easy rise, the way gradually grows steeper and steeper. It takes the surveyor by surprise, when, on ground that looks to be almost level, his instruments tell him that he is 40, and then 50, and then 60, ft. higher at the end of a mile than he was at the beginning. At 95 miles, he has risen to 5 500 ft. Then, suddenly, in the distance of 5 or 6 miles, as the wagon road runs, the mountain has lifted itself 2 500 ft. higher still, up to the *cumbres* or sharp edge of the table-land which stretches away in a nearly level plain for 400 miles. This abrupt mountain wall was the railroad problem.

"*Ravines to be Crossed.*—There were also difficulties from deep and steep ravines or *barrancas* which cut across the path. One in particular, the Barranca de Metlac, hid a rushing stream 300 ft. below the surface of the surrounding fields. There was also a line of little hills, the Chiquihuites, standing as a barrier directly across the line of approach. Such were some of the natural obstacles, and it was evident that the best professional advice would be essential to success.

"*An American Engineer and his Party.*—This help was secured by the engagement of an eminent and experienced American engineer, Capt. Andrew Talcott. He was a graduate of West Point, and had rendered conspicuous service in the United States army for 20 years. During that time he had been engaged on such works as the fortifying of Hampton Roads, the building of Fort Hamilton, in New York Harbor, the work at Fort Adams, in Rhode Island, and the improvements of the Upper Hudson River. Resigning his army commission, he had been in demand for boundary surveys in Maine, Michigan, and Iowa, and had been chief engineer of two important railroad systems. He was well fitted by training and experience for this new task.

"A surveying party of some thirty men was at once recruited in the neighborhood of New York, several of them being taken from the staff of the Croton Water-Works and others from the Pennsylvania and Reading Railroads. The chief assistants were M. E. Lyons, of Reading, Pa., and R. B. Gorsuch, of New York, the latter having already had experience in Mexico in connection with the building of the experimental railroad to Guadalupe. Camp equipment, instruments, etc., were shipped from New York.

"The main party sailed by steamer from New Orleans, on January 1st, 1858, and landed at Vera Cruz 4 days later. A camp was immediately established at Tejeria, 9 miles out from the city, and the working parties were organized. These were three in number, a headquarters party, to explore the country on horseback, and two field parties, each with transit and level, prepared to do exact work and keep careful records. In view of the uncertain character of strangers who might be abroad, the camps were always guarded by a sentry at night, and by day each American wore a revolver in his belt.

"*Roads and Routes of Travel.*—At that time, in Mexico, as in our own colonial days, land travel, if not done on foot, was chiefly on horseback. Merchandise was packed on mules. Wheeled vehicles, except in the cities, were few. Although there were trails and bridle paths in every direction, there were no roads possible for carriages except the one thoroughfare, the *camino real,* from one large city to the next. From Vera Cruz inland there were two such highways. One was by way of Jalapa, mounting the table-land on the north side of Mount Perote, beyond the great snow peak of Orizaba. This route was chosen by General Scott with the American army in 1847. The other road, somewhat more direct, passed through the City of Orizaba, climbed the mountain pass at Aculcingo to the south of the snow peak, and joined the Jalapa road at San Marcos, many miles to the west, leading on thence to Puebla and Mexico. This was the route taken by Cortez and his Spaniards. It was the regular stage route in 1858, and was adopted as the base of the survey. In many respects this was a remarkable road, built probably by the Spaniards 200 or, perhaps, 300 years ago. The location of this broad highway through a mountain country showed great skill in the builders. The massive, arched stone bridges were still intact, and as strong as when first built. There were in many places the remains of a stone pavement; but years or centuries of neglect and spoliation had left only scattered blocks of granite which were more of a hindrance than a help to passage. Along this national road, mule trains with packs of merchandise were constantly passing, up and down, but almost the only wheeled vehicle that used the road was the *diligencia* or stage coach which passed daily in both directions.

"The journey from the coast to the capital required 3 days. At Vera Cruz the coach—one of the familiar type which we know as a Concord coach—was placed on a platform car and went by train 9 miles to Tejeria. At that point horses were attached. There were never less than six horses and, on the steeper grades, eight and ten were often used. Relays were taken every 10 miles and, where the going was good, the team was driven at a gallop. The first day's trip was 82 miles to Orizaba, arriving in time for dinner, a quiet evening, and a part of a night's rest. Starting again before daylight, a distance of more than 90 miles to Puebla was made on the second day. The third day's ride of about 80 miles, passing over an elevation of 10 500 ft. above the sea and thence running rapidly down, brought the tired passengers into Mexico in the afternoon.

"*Robbers on the Road.*—The journey was tedious enough, but the magnificence of the scenery in the mountain passes was to many travelers no slight compensation. The trip was also subject to some thrilling adventures. The road, especially that part of it between Orizaba and Puebla, was infested with bands of robbers. It was the common daily experience of stage passengers to be stopped once or twice in the early morning and invited, with profuse apologies but at the point of a pistol, kindly to alight from the coach, while three or four men coolly proceeded to possess themselves of money or any valuables that might be found on the persons of the travelers or concealed under the cushions. Articles of clothing, such as hats, coats, and shoes, were often taken from the wearers, and silk dresses

were sometimes stripped from women. If so fortunate as to find a priest among the passengers, the bandits would persuade him also to contribute by holding a pistol at his head until he pronounced a full absolution for the crime.

"The leaders of these robber parties were often sons of wealthy landowners who found life dull on the *hacienda* and led these forays for the mere excitement or the fun of doing it. Passengers, on the other hand, went prepared to pay a little toll, considering it a custom of the country, and carried only a few silver dollars and an old silver watch, and wore their old clothes when compelled to travel by diligence. The American surveying party, whether in its own camps or at its work, was not in any way molested. In September, 1858, however, when the preliminary survey was completed, a party of nine Americans found themselves paid off at the City of Mexico and were unable to obtain New York funds, except at a discount of 15 per cent. They therefore decided to engage the coach for themselves and, being well armed, to carry some $3 000 in coin to the coast. On the second day out, they were twice challenged in the usual way by robbers, and a lively fusillade followed in each instance. No one of the American party was touched, but the woodwork of the coach showed more than one bullet hole. One of the assailants was left dead by the roadside and two others were afterward known to have died from wounds received.*

"A surprising sequence of the above encounter was the sad fate of a Mexican who was traveling in the opposite direction on that unlucky day, and was alone in the coach when, along in the afternoon, it reached the spot where the American engineers had been in the morning. This solitary traveler, who had never heard of the engineers, was taken out by a band of ten men armed to the teeth, was stripped to his underclothes, tied to a tree and lashed without mercy. This was Mexican retribution.

"*Disturbed Conditions.*—The work of the two field parties, begun early in the year, had not continued 2 months before it became evident that the disturbed political and military conditions must seriously embarrass their work. Comonfort was President, and Zuloaga was contesting his claim. Detachments of troops from one side or the other were marching back and forth over the road, and in a small way were fortifying ground here and there along the line. Now and then the sound of cannon and volleys of musketry was heard. There was no very serious clash of arms, but there was just enough of the form of fighting to be annoying to peaceable surveyors. Consequently, the force was reduced, one field party was kept, and the others returned home. The remaining party then began in earnest to examine the most critical point on the line, the abrupt ascent of the *cumbres.*

"*A Choice of Mountain Passes.*—The stage route climbed the mountain by the pass at Aculcingo by a series of ten or twelve extended zigzags. A railroad could hardly do that. After one height was reached, there was another beyond it, and the attempted line of survey became hopelessly involved in a very tangle of hills. A month was

* A notice of this affair may be found by the curious in *Harper's Weekly* for October 23d, 1858, at page 679, under the heading of "Mexico; A Band of Robbers Punished".

spent in trying the possibilities of the ground, and the result was failure. Then the scouting party reported that a few miles to the north was another pass up from Maltrata, and, on the further side of it, a very long mountain spur thrust out for miles into the lower country, a kind of buttress to the table-land. Thither the party turned. They found no carriage road in the Maltrata Pass and the zigzag turns for horses and mule trains were even more numerous than at Aculcingo, but hope was fixed on that prolonged spur of the mountain the sides of which towered half a mile above the village and were as yet unexplored and covered with low trees and tangled brush. After climbing up the path to Boca del Monte, the 'mouth of the mountain', which stands at an elevation of 7 922 ft., the transit was set at a downward gradient of 200 ft. to the mile, and, with only this as a guide, the mountain side was explored. No account was taken of curves, bridges, or tunnels. The chopping out of the path was slow. Straight lines and their angles were carefully measured and noted. The leveling instrument was close behind. Another full month was spent creeping farther and farther along.

"At one point, now called the 'Devil's Balcony', the cliff is so precipitous that it seemed to hang above the streets of Maltrata Village, 2 000 ft. below. Winding on and on, the point of the spur was turned, and a transverse valley to the left was entered. At the end of that came another turn on a hillside to the right and, at some distance farther, another and another turn, until the line drawn on paper resembled the outline of a boot with the leg resting along the valley. At last, after 11 miles of sharp descent, the low level of the village was successfully reached; but there was more of it to come. The water drainage of the valley is through a narrow gorge, so wild that the local name for it is the *Barranca des Infiernillos,* or, in English, 'The Ravine of little Hell'. Down this rocky gorge the line goes plunging on, turning and twisting again, until at last, in the Valley of Encinal, it comes out into the midst of smooth fields and 10 miles of a straight course brings the line to Orizaba. In 18½ miles, the descent is 3 000 ft. The average of that is 165 ft. to the mile, but many miles are on a grade of 200 ft. or even more. This route has now been in use for 40 years. The locomotives are of the twin, Mogul type, built by Fairlie, each with two boilers, which alternate in supplying steam for the desperate pull.

"*Across the Deepest Gorge.*—The second problem was the crossing of the Barranca de Metlac. As a straight line across was only 1 400 ft., it was a challenge to the engineer to throw a suspension bridge over it, or at least a tubular or cantilever structure with high piers and an enormous span; but the first cost of such a work would also be enormous, and the cost of maintenance very great, and neither of those things was done. Perhaps, if the steel construction of high piers had been as well understood 50 years ago as it is to-day, the straight line might have been secured; but, instead of that, the line, crossing the edge of the ravine, runs down along the inside bank on a narrow shelf for a mile or more up stream until it reaches a broader part of the ravine and then turns across on an almost semicircular bridge, about 100 ft. above the water, and goes back by a similar shelf on the farther side until it reaches the top again and continues on its way.

"The third point of difficulty at the Chiquihuite is met by a considerable detour away from the line of the highway, the railroad striking the side of the hill and rising until it joins the carriage road again, crosses it and passes at once by a high bridge to the farther side of the Atoyac River.

"Thus, step by step, each problem was successfully solved, and in September, after a little more than 8 months in the country, the party returned to New York, the line of survey was plotted, and the report made.

"*Further Delay and Final Success.*—For a long time, nothing more was done. A new grant from the Government was made in 1861. The French Army entered Mexico in 1862. To facilitate their movements they contracted with Escandon to build portions of the road. It was extended over the lower, open country to Paso del Macho, 48 miles from Vera Cruz. It was also carried from Mexico east to Puebla. During this foreign régime, 133 miles were constructed. A still more liberal contract was made with the Republican Government in 1867, and another in 1868. The cost of building was found to be unusual and extreme. Incredible as it may seem, the Government insisted that the road should be built from both terminals simultaneously, making it necessary to haul heavy material, machinery, and equipment hundreds of miles over rough mountain roads in order to build back from the capital. The work of construction of the mountain division, still under the supervision of Capt. Talcott, was taken up in earnest in 1864; but Escandon was compelled to seek outside pecuniary help, which he found in England, and it was an English company which finished the road in 1872—the first railroad in Mexico.

"In spite of its great cost, the need of the railroad has been so great that on the $46 000 000 invested good profits have been paid out of the heavy charges, at least up to within the past year. The road is under English management, and the impression is common that the English built it. The fact is that the survey, the location, and the plan of construction were American throughout.

"From the character of the road it will be seen how easily it can be obstructed by acts of war, by the destruction of bridges and blocking of tunnels; but let us hope that this will never be done. The road is itself a triumph over natural obstacles, it ought not to be ruined by human passions.

"ALBANY, N. Y., JUNE 19TH, 1914."

APPENDIX D

CONSTRUCTION OPERATIONS, 1861–64, FRENCH OCCUPATION.

The unstable conditions existing in Mexico at the time of the presentation of Capt. Talcott's report, which prevented for the time being the construction of the proposed railroad, have already been noted. Under date of Tacubaya, May 19th, 1859, Don Manuel Escandon wrote Capt. Talcott:

"I have set my shoulder to the wheel, and have no doubt of procuring the formation of a Mexican and Puebla Company, to commence at once the works between these two cities, crossing by the Llanos de Apam, which line later on would be continued from Puebla to Orizaba and thence to Vera Cruz."

The question of a joint intervention in Mexican affairs by the European powers of Great Britain, France, Spain, and Prussia was mooted in 1860, and on December 14th, 1861, a Spanish fleet appeared before Vera Cruz, which was at once occupied by the Spanish troops under General Prim. The French arrived soon after, and in September, 1862, more French troops arrived. In view of the unhealthiness of Vera Cruz, the convention of Soledad was concluded with the Mexican Government, permitting the foreign troops to advance to Orizaba.

No construction work was done by Señor Antonio Escandon under the grant of May 31st, 1857, and on April 5th, 1861, he obtained a new grant from the Mexican Government which materially modified the provisions of the former one.

Capt. Talcott received a call to return to Mexico in May, 1861, with which he was unable to comply, and suggested the employment meantime of Mr. M. E. Lyons, his former Principal Assistant, which was accordingly done, and Capt. Talcott did not return to Mexico until March 1st, 1862, having left Charleston, S. C., on January 9th, 1862, on the blockade runner *Carolina* for Nassau, and thence *via* Havana to Vera Cruz, where on arrival he resumed his duties in connection with the construction of the Mexico and Pacific Railroad.

The French, on their arrival in Mexico, found no railway, except the few miles of track operated by mules from Vera Cruz, which was used to transport passengers and their baggage about 9 miles to Tejeria, from which point transportation to the interior was over old Spanish highways that at many points were almost impassable during the rainy season; and their efforts were immediately directed toward the construction of a railway across the *Tierra Caliente,* in which they seem to have had the co-operation of the "Escandons", for entries found in Capt. Talcott's diary for February, 1863, indicate that, as

the representative of Antonio Escandon, he was supervising work in progress in Soledad.

It appears, therefore, that the demand for better facilities for transportation by the French Army was responsible for the construction of what is now known as the "Mexican Railway" from Vera Cruz to Soledad, where it crosses the Jamapa River, and practically determined the Orizaba Route for the Imperial Mexican Railway Company in 1864.

It was not until after the arrival of Emperor Maximilian (Vera Cruz, May 28th, and City of Mexico, June 19th, 1864) that Antonio Escandon made rapid progress in his negotiations with English capitalists which finally resulted in a contract, dated August 19th, 1864, between "The Imperial Railway Company, Limited", and Smith, Knight and Company, Limited, by which the last named Company undertook to

"complete and construct a Railway from Vera Cruz to the City of Mexico and a Branch Railway to Puebla, according to the plans, sections, and specifications in that behalf made by Mr. Talcott, one of the Engineers of said Company."

In anticipation of the conclusion of this contract, Don Antonio Escandon had written to Capt. Talcott as follows:

"Andrew Talcott, Esq.,
Mexico. "Paris, 31 July, 1864.

"Dear Sir:—I thank you for the information you give me concerning your operations on the portion of the line near Mexico and am happy to see that Messrs. Almazan, Merino and your son Richard are employed on the same. You must bear in mind, as I told you before, that no rails are to be placed on that part of the line and that you have nothing else to do but to go on with the earthworks without exceeding your estimate of 1858, nor anyhow the amount of $15 000 per month. Any such excess would be a dead loss for me; on the contrary, I would profit by any difference below your said estimates, in conformity with which these works will be reimbursed by the contractors.

"I must renew my observations concerning the settlement of account with the French Government should the question be brought before you. This settlement must be done in strict accordance with the contract made with the French Engineers. That is to say, from the sums advanced must be deducted the amounts received by the French and the customs dues up to Dec. 31, 1864, and those due to the Railway for carriage of the French Army, its baggage, etc. If any difference results against me, I will pay it forthwith in money on 1st January, 1865, and can admit of no settlement made on other terms than these.

"I will besides recommend you most particularly to reserve all my rights against the French Government in consequence of the losses sustained by the Company by cases of *force majeure,* such for instance as that which occurred when W. Lyons was mortally wounded by the

guerillas, who carried off a certain sum of money belonging to the Company and destroyed part of the road. (The illness of W. Lyons, as you know, delayed for some time the progress of the works), besides several other cases which are not present now to my memory.

"The principal injury we sustained was when the guerillas carried off all the workmen on the line with the cash in hand for payment of the same. This is the point that must be principally insisted upon.

"Yours,

"ANTONIO ESCANDON."

After the execution of the contract with Smith, Knight and Company, Don Antonio Escandon wrote a letter to Capt. Talcott, as follows:

"PARIS, August 31, 1864.

"Messrs. Smith, Knight and Co., the constructors, have had the good fortune to secure the services of Mr. Lloyd, a friend of Messrs. Gibbs, who has constructed an important Railway in Peru, where he has resided 10 years. This gentleman, with Mr. Samuel, will leave for Mexico probably by the St. Nazaire Steamer of September. He will remain only a short time there to see what are the materials that he has to purchase in Europe for account of the constructors, after which he will return to England. You must therefore get everything ready for the delivery of the constructed part of the line, which will be made immediately after the arrival of these gentlemen.

"I have asked the French Minister of Public Works to grant me an audience in which I intend to request him to withdraw entirely the French Control in consequence of the formation of the new Company. So I think you will soon be easy on that score.

"THURSDAY, 1st September, 1864.

"After writing what precedes, and at the last hour I receive your favor of the 29th July.

"By what I have already told you, you will see that it is very urgent that you should immediately go over to Vera Cruz to get the Schedule made of all the Stock belonging to the Railway, without distribution even of the light rails, which, by my contract, will be returned to me by the Company.

"My engagement being to deliver the road up to the Paso del Macho I need not tell you that no expense for Surveys, etc., must be made beyond that point.

"ANTONIO ESCANDON."

It is very evident from the foregoing that what is now known as the "Ferrocarril Mexicano" from Vera Cruz *via* Soledad and Cameron to Paso del Macho was built by Don Antonio Escandon under the concession for the "Mexico and Pacific Railroad," for which surveys were made by Capt. Talcott in 1858.

It also shows that this part of the road was built under the control of the French Government, in accordance with an agreement made

with French engineers, and that for this 77 km. of railway the Imperial Mexican Railway Company paid Don Antonio Escandon an agreed price after it had been completed at his cost from Vera Cruz to Paso del Macho.

It also appears that Don Antonio Escandon constructed, in part at least, the line from the City of Mexico to Guadalupe and east of Guadalupe to the Llanos de Apam, for which work he was paid by Smith, Knight and Company, on the basis of Capt. Talcott's original estimate of its cost, for the construction of this part of the line was included in the Company's contract to build the railroad from Paso del Macho to the City of Mexico.

APPENDIX E

Construction Operations, 1864–66, Under Emperor Maximilian.

Immediately after the organization of the Imperial Mexican Railway, August 19th, 1864, extensive measures for the speedy construction of the unfinished portion of the railroad were begun, although the transfer of the concession held by Don Antonio Escandon was not approved by the Emperor Maximilian until January 26th, 1865, which approval fixed the time of completion of the entire line by April 30th, 1869. The chief contractors, Messrs. Smith, Knight and Company, were the builders of the Chilian railroad from Valparaiso to Santiago, and came from that country to Mexico. They had their own staff of engineers, Mr. S. Samuel, Chief Engineer in London, England, and Mr. William Lloyd, Chief Engineer in Mexico, Messrs. William Cross Buchanan and Alister Fraser, Division Engineers, Mr. Richard Ingalls, Superintendent, and others, all Englishmen. On the staff of Capt. Andrew Talcott, were his son, Mr. Richard H. Talcott, now of Albany, N. Y., Mr. T. M. R. Talcott, now of Richmond, Va., Sebastian Wimmer, M. Am. Soc. C. E., now of Albany, Minn., the late Gen. W. H. Stevens, formerly on the staff of Gen. R. E. Lee, Col. H. T. Douglas, now of New York City, and other American engineers.

Many of the latter had served in the Confederate Army during the Civil War (United States), and had secured positions in Mexico at the close of that war, in April, 1865. The chief contractors, Smith, Knight and Company, were afterward succeeded by George B. Crawley and Company. Mr. Sebastian Wimmer, after his arrival in Mexico, was placed in charge of the construction of the seven upper sections of the Maltrata Incline (so-called) on April 26th, 1865, and some time after was placed in charge of additional sections, lower down, near the Town of Maltrata. While in charge of this work he made a number of modifications and changes in the alignment which greatly improved it.

The following excerpt from Mr. Wimmer's diary is of more than ordinary historical interest:

"*Saturday, April 29, 1865.*—Emperor Maximilian arrived at Maltrata at 12 o'clock noon. I was introduced to him, as well as the other engineers, and had quite a chat with him. He wanted to know how I, a German, from Munich, Bavaria, Germany, came to be associated with the English party of engineers engaged on the Imperial Mexican Railway. I explained that I was a citizen of the United States, a practicing civil engineer for the past fourteen years (1851-1865), having arrived in New York city, June 2d, 1851, and when civil engineers from the United States were required for service in Mexico, I applied for a position on this railroad and then received an appointment on this important work.

"He told me that he well remembered Abbot Boniface Wimmer, my uncle (with whom I arrived in New York as above), and favored his project to establish in America 'The Benedictine Order'.

"We all had a fine dinner in the open verandah of the chief contractor's house (Smith, Knight & Co.), a native Indian brass band played in the open patio. Leo Thun, Major General of the Austrian troops, quartered at Puebla, was one of his party, also his private secretary Elvin, Minister Ramista, Prince Wittkenstein, a Russian officer, and others of his escort. Others present were Wm. Lloyd, Chief Engineer of Smith, Knight & Co., Mr. Richard Ingalls, Superintendent, Engineers Richard H. Talcott, Fred. Simons, my friend Rudolph Wieser, Mr. Wm. H. Burr, Dr. Manifred, Wm. Cross Buchanan, John Quinn, and others.

"A messenger from Orizaba arrived with a telegram for the Emperor, announcing that President Lincoln of the United States was killed on April 14th, 1865, in Ford's Theater, Washington, and Secretary William H. Seward and Fred'k. Seward wounded, by John Wilkes Booth, the assassin. This news at once stopped the dinner, as the Emperor wanted to put himself in telegraphic communication, and Orizaba was the nearest point, some 15 miles away. We all escorted the Emperor to Orizaba that afternoon."

Also this excerpt:

"*Monday, December 25, 1865.*—At Orizaba. Christmas Day. Hot and dry. Attended midnight high mass with Davison. At 8 A. M. all we engineers rode about 3 miles down toward Cordoba, to escort Empress Carlotta to Orizaba, who was returning from Yucatan to the City of Mexico. She arrived about 11 o'clock, seated in an open carriage with a court lady, and accompanied by a large number of French Zouaves on horseback, several hundred, plowing through six inches of dusty road, causing such a dust that we could not see one another, and covering the poor Empress and her lady companion so that it was awful to behold. They escorted her to the Cathedral for a grand reception by the authorities and thence was to receive us, but, being exhausted, she went to Pringas residence to rest and to receive us the next day.

"*Tuesday, December 26, 1865.*—Hot weather. I called with Ed. Melgar on Col. Roderigues to arrange about our introduction to the Empress. She received us at Pringas. General Uraga (one leg lost in war lately) acted as introducer. Richard Ingalls was our spokesman. Richard H. Talcott and myself, Mason Peek, Maynadier, W. S. Davison, Wm. B. Cooper, and Slate were introduced. She spoke Spanish fluently, but she soon discovered that most of us were rather embarrassed and preferred English, and when Mr. Ingalls was not aware that the Empress spoke English, she answered that it was the second language she had learned and loved to speak it. We thence got along very well, and when Mr. Ingalls invited her to come on her way up to the city of Mexico *via* the *Las Cumbres* work, she replied that she loved to accept the invitation, as the Emperor had already told her of his delight of having been there last spring, but she had already received a telegram from the Emperor, who was in Puebla, to

come on at once, without further delay, and as a dutiful wife, 'I must obey' and thus be unable at this time to see for herself how far the contractors of the railroad had proceeded toward a soon completion, for you have seen what a terrible thing it is to travel in this country, and so exhausting, and asked us to hurry the work to completion."

On July 8th, 1866, the Empress Carlotta left Mexico on a visit to Napoleon, Emperor of France, to arrange Mexican affairs, never to return. Her husband, the Emperor Maximilian, as is well known, was executed June 19th, 1869, at Queretaro.

In the spring of 1866, the Imperial Mexican Railway Company became financially embarrassed, and Capt. Talcott had to obtain funds from Barron, Forbes and Company, of the City of Mexico, to meet engineering expenses in carrying out instructions he had received to prepare and submit final estimates of the work done so far by Smith, Knight and Company and their successors George B. Crawley and Company.

At that time the railway was in operation between Vera Cruz and Paso del Macho, 48 miles, and was opened to Apizaco, 87 miles from the City of Mexico, for freight service, January 21st, 1867.

APPENDIX F

Construction Operations, 1867–73, Under Republic of Mexico.

Capt. Talcott severed his connection with the Imperial Mexican Railway on February 10th, 1867, he holding at the time the position of Joint Engineer in conjunction with Mr. James Samuel, of London, England. Leaving the City of Mexico, February 23d, 1867, he arrived in New York City on April 5th, 1867. During his connection with this railway he had constructed 135 miles of road.

After the downfall of Maximilian and the withdrawal of the French Army, Señor Don Escandon applied to the re-established Government of the Republic, to revalidate his concession, and, notwithstanding he had been a staunch supporter of the French and of the enemies of the Republic, the Mexican Government, on November 27th, 1867, made a new contract with him, under more liberal terms than his former grant. This contract was again modified on November 11th, 1868, and the railroad was finally finished on December 31st, 1872, under the Escandon grant, 16 years after his first contract was made, and 36 years from the date of the first concession. It was formally inaugurated on January 1st, 1873, by President Sebastian Lerdo de Tejada.

The Puebla Branch was finished on September 16th, 1869, the main line to Paso del Macho, in 1864, to Atoyac, January 9th, 1871, to Cordoba, August 22d, 1871, to Fortin, December 8th, 1871, and to Orizaba, September 5th, 1872.

A great deal of misinformation has been published about the Mexican Government requirement of hauling rails and equipment overland. This requirement was a practical necessity, and was dictated by common sense, as the construction work on the table-land was comparatively light and the grading between the City of Mexico, Apizaco, and Puebla was finished 5 or 6 years before that of the sections lying farther toward Vera Cruz, between Boca del Monte and Paso del Macho, where the heaviest work on the railway occurs.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

COHESION IN EARTH: THE NEED FOR COMPREHENSIVE EXPERIMENTATION TO DETERMINE THE COEFFICIENTS OF COHESION

By William Cain, M. Am. Soc. C. E.

To be Presented February 2d, 1916.

Synopsis.

The ordinary theory of earth pressure pertains to a granular material, such as clean sand, gravel, or rip-rap, supposed to be endowed with friction only. For ordinary earth, and particularly with clay, experiments given show that, in addition to friction, the earth is endowed with cohesion, and the practical importance of determining experimentally the simultaneous coefficients of friction and cohesion for every variety of earth is earnestly urged.

The results of the few experiments made on various earths and clays are given, and attention is called to the small values of the coefficients of friction and to the large values of the coefficients of cohesion in the case of consolidated earth and clay.

Harking back to the laws of friction and cohesion, first formulated by Coulomb, a simple form of apparatus for experimenting is given to illustrate principles, though it is realized, that when the earth is subjected to great pressures, a more complete testing machine is

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

desirable. In fact, a standard machine to which engineers could send samples of the earth to be tested, seems desirable.

The results of the more recent experiments made, in both France and England, are, in some respects, so unexpected and significant that a mere glance at the figures will show the immediate need of a comprehensive system of experimenting, not only to determine the true laws of friction and cohesion of earth, but likewise to determine the coefficients of friction and cohesion for ordinary earths and clay, in various stages of consolidation.

Coulomb, about 1780, was the first to formulate the laws of friction and cohesion, as affecting a mass of earth. For a homogeneous earth, these laws may be stated thus:

(1) The maximum frictional resistance that can be exerted along any portion of a plane in the interior of a mass of earth equals the normal pressure, P_n, on the portion of the plane considered, multiplied by f, the coefficient of friction, where f is a constant for the earth considered.

(2) The maximum cohesion equals the area of the portion of the plane considered, multiplied by k, the coefficient of cohesion, or cohesion per unit of area, where k is a constant for the particular earth in question. Thus, if A denotes the area of the portion of the plane under compression and Q the total resistance to sliding along this interior plane, then,

$$Q = P_n f + k A \qquad (1)$$

A great number of experiments have been made to determine f, the coefficient of friction for various materials, on the supposition that k was either zero or negligible; but very few have been made to determine, at the same time, the coefficients of cohesion.

A limited number of experiments, with this object in view, have been made by Collin (1846), Leygue (1885), Jacquinot and Frontard (1910), and A. L. Bell (1914). With the exception of Collin, the experimenters determined the coefficients, f and k, from the same set of experiments. Although the apparatus used by one experimenter was not the same as that used by any other, yet, in principle, the following simple device can be supposed to represent the method of

experimenting. In Fig. 1, a thin slice of earth is supposed to be placed between two metallic plaques, which are rough on the inside. These are then firmly pressed together, but without contact, and the resistance to the relative displacement of the two plaques is then measured.

Suppose the pressure to be due to a weight, and let P_n represent the sum of this weight and that of the earth and plaque above the horizontal plane of shear, AB; also, let Q be such a horizontal force applied (through a cord) to the upper plaque in the plane, AB, that sliding of the earth above the plane, AB, over the earth below it is "impending", which means that the slightest increase in Q would cause actual sliding. The earth along the plane, AB, of the area, A, resists the force, Q, by the friction and cohesion acting to the left along AB; hence, for equilibrium, at the instant motion is impending, we have,

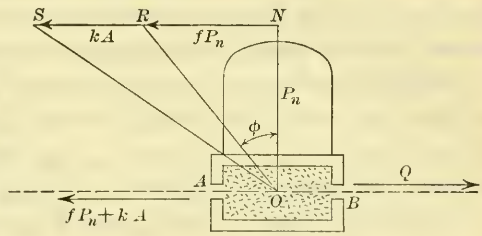

FIG 1.

$$Q = f\, P_n + k\, A.$$

The normal reaction of the earth just below the plane, AB, $= P_n$; hence, if $ON = P_n$ is laid off vertically upward, and $NR = f\, P_n$ and $RS = k\, A$ are drawn horizontally to the left, then $NS = f\, P_n + k\, A = Q$. Also, if ϕ denotes the "angle of friction" of the earth, then $f = \tan. \phi$; so that the angle $NOR = \phi$. Note carefully that, in earth endowed with both friction and cohesion—coherent earth, as it may be styled—the "angle of friction" is not the surface "angle of repose." The latter is always greater than ϕ, a definite mathematical relation existing between the two angels and the depth of embankment.

If we designate by p_n the normal pressure per square foot on the plane, AB, and by q the total resistance to sliding per square foot of the plane, then, by dividing both sides of the preceding equation by A, we obtain,

$$q = f\, p_n + k = p_n \tan. \phi + k \ldots\ldots\ldots\ldots\ldots (2)$$

where q, p_n, and k will be expressed in tons, or pounds per square foot, according as Q and P_n are expressed in tons or pounds. As k and $\tan. \phi$, for any particular homogeneous earth, are constants, if the variable normal unit pressure, p_n, is laid off, from the origin, along

the axis of abscissas, then the corresponding value of q can be represented by an ordinate, and the equation is that of a straight line, making the angle, ϕ, with the axis of abscissas and cutting the axis of ordinates at the distance, k, above the origin. From the results of the experiments, simultaneous values of p_n and q can be plotted, and an average line drawn; otherwise, corresponding experimental values of p_n and q can be substituted in an equation of the foregoing type, and, from the resulting equations, the most probable values of tan. ϕ and k can be found by the method of least squares.

It may be said that it is still an open question, whether friction and cohesion, according to Laws (1) and (2), are both exerted at the same time, but the limited number of experiments which have been made seem to justify the assumptions and to verify approximately Equation (2). It seems probable, however, for very compressible substances, such as fresh earth, especially if pulverized or in lumps, that the coefficient of cohesion should increase with the normal pressure; for the area of the actual contact of the particles increases with the pressure, because such pressure squeezes the particles together and causes a more intimate contact; hence, since, by Law (2), the cohesion varies directly as the area of contact, it should prove, for such earths, the greater the larger the normal pressure.

This objection applies to a much less degree to a slice of earth cut out of a bank and experimented on in its virgin state, just as it was in the bank. However, if fresh, clayey earth is taken from a new embankment, the earth being more or less pulverized, the coefficient of cohesion is doubtless small; but if such earth is thoroughly wetted and rammed, so as to approximate to a puddle wall, its cohesion is very much increased, as the contact of the particles is more intimate than when the earth was in a friable state.

From similar considerations, it would appear reasonable to suppose that the unit cohesion in a bank of earth should increase with its depth, so that it is highly desirable to subject the earth, between the plaques, to pressures corresponding with those actually sustained in banks, say, up to 50 ft., or more, in height, in order to ascertain the variation, if any, in the coefficient of cohesion. If this variation with the height is appreciable, then an average value of the coefficient, for a particular height of bank, will have to be assumed for the imaginary

homogeneous earth to which the theory pertaining to coherent earth strictly applies.

It may be remarked, further, that Equations (1) and (2) are only valid when no relative motion of the plaques occurs. When motion once occurs, it would seem that the cohesion of the earth along AB would be destroyed, and that only friction is exerted. This is analogous to the case of the trench, in which constructors are very solicitous about placing the bracing before a break in the earth starts, for in such case, the cohesion along the surface of the earth is lost, only friction remaining, so that the pressure on the bracing is very much increased. If this reasoning is true, is not the "friction of motion" between any two bodies (wood, iron, stone, etc.), more nearly the true friction than the "friction at rest"? It is possible that cohesion as well as friction may be exerted between any two bodies at rest, so that expressions of the form of Equation (1) should be written to correspond to the results of experiments made as previously indicated. The solution of such equations will determine f, and also k, if it is not zero, as usually assumed.

The results of the experiments on earth to determine the coefficients $f = \tan. \phi$ and k, after a method equivalent in principle to that just outlined will now be given.

In the experiments of Leygue,* the normal pressures were very small—only from 7 to 40 lb. per sq. ft.—but the laws of Coulomb were practically verified within these narrow limits. The results were:

Dry sand...........	$f = 0.70$	$\phi = 35°$	$k = 1.47$ lb. per sq. ft.
Wet sand..........	$f = 0.85$	$\phi = 40° 22'$	$k = 8.28$ " " " "
Very wet sand.....	$f = 1.70$	$\phi = 59° 30'$	$k = 6.36$ " " " "
Damp fresh earth...	$f = 1.63$	$\phi = 58° 28'$	$k = 18.45$ " " " "

Leygue states that Collin found, by an independent method, that for clayey earth, $k = 23.1$; and that for clay of little consistency, $k = 39.5$ lb. per sq. ft.

From the small value for cohesion for dry sand (only 1.47 lb. per sq. ft.), Leygue seemed to be warranted in ignoring it in analyzing the results of his carefully conducted experiments on retaining walls and boards. As the results did not agree very well with those of the

* "Nouvelle Recherche sur la Poussée des Terres," *Annales des Ponts et Chaussées*, 1885, Part II, p. 788.

sliding-wedge theory, ignoring cohesion, the writer reviewed the subject of experimental walls in an extended paper,* reaching the following results:

For walls of a few feet in height, backed by dry sand, the ordinary sliding-wedge theory, ignoring cohesion, agreed in its results fairly well with those of experiments, provided the friction between the earth and wall was included from the start; but that the Rankine theory was not sustained generally by the experiments. In Leygue's experiments, however, the walls or boards were only a few inches in height, and it was found that the influence of cohesion was marked and that the results could only be harmonized with theory by including the influence of cohesion as well as that of friction. The cohesion required was very small—only about 1 lb. per sq. ft., which is a little less than that found by direct experiment—but its influence on the results was marked, owing to the small height and consequent small weight of the prism or wedge of rupture. In the course of the investigation, a complete graphical method was devised, which can be applied in ascertaining the thrusts against retaining walls and trench bracing for the coherent earth supposed. In such applications, it is absolutely necessary to find the coefficients, f and k, by the method previously detailed. In fact, the value of ϕ for consolidated earth is found to be much less than the usual so-called "angle of repose", and its low experimental value, as given in the remaining results, will doubtless give a decided shock to those not familiar with this recent experimenting.

The results of the experiments made in 1910, by MM. Jacquinot and Frontard,† on earth taken from an earthen reservoir dam which had failed owing to a considerable lowering of the water level, will next be given. The dam was constructed in the most approved manner, of the best materials, the composition of the earth tested being 60% clay, 32% silica, as an impalpable dust, and 8% silicious sand. Water was added to the earth, and it was then kneaded and compressed with the hands so as to make a firm though pasty cake, which was then inserted between the plaques. Not enough water was added to cause a lateral flow when the cake was under compression. The details of

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* "Experiments on Retaining Walls and Pressures on Tunnels", *Transactions*, Am. Soc. C. E., Vol. LXXII (1911), p. 403.

† Given in some detail by Professor Résal in his work, "Poussée des Terres", Part II, "Theorie des Terres Coherantes", p. 327, Paris, 1910.

the apparatus used are not given. The results for the first series of experiments are given in Table 1.

TABLE 1.

p_n, in pounds per square foot.	$f = \tan. \phi$. $\phi = 8°$.	k, in pounds per square foot.
692	$f = 0.14$	395
2 980	$f = 0.14$	420
5 665	$f = 0.14$	403
7 154	$f = 0.14$	448

The earth in the bank weighed 112 lb. per cu. ft., so that the recorded pressures corresponded to the vertical pressures experienced at depths of from 6 to 64 ft. for the earth in question.

After the first series of experiments had been performed, the manometers got out of order, so that a correction had to be applied, and, as there was some doubt as to the accuracy of the results, they will not be given. From all the experiments, however, some referring to the earth as it was taken out of the bank, and some to rammed earth, the quantity of water being varied, it seemed that the following conclusions, as stated essentially by Résal, could fairly be drawn:

(1) Coulomb's laws were approximately verified.

(2) $f = \tan. \phi$ ranged only from 0.14 for a soft pasty earth to 0.18 for the earth nearly dry. Neither the quantity of water used, nor the ramming or puddling, caused much variation in f.

(3) On the other hand, the quantity of water used affected the cohesion very much, and sufficient ramming could more than double the coefficient of cohesion.

The large values of k were to be anticipated, as the theory of open cuts and trenches would lead one to expect, for ordinary consolidated earth, values of k running into several hundred pounds, but the low values of ϕ (only from 8 to 10°) are somewhat startling, and indicate that our ideas with respect to the coefficient, f, and earth-pressure theory generally, may have to be considerably revised.

The values of k and ϕ, given in Table 2, lead to the same conclusions. They represent the results of experiments on clay, in its virgin state, as taken at various depths from an excavation carried

below low water, in the course of sinking a large number of monoliths which are to form the foundation of the outer sea-wall of a dockyard at Rosyth.*

The figures given represent fair average values for k and ϕ.

TABLE 2.

	k, in tons per square foot.	ϕ, in degrees.
Very soft puddle clay	0.2	0
Soft puddle clay	0.3	3
Moderately firm clay	0.5	5
Stiff clay	0.7	7
Very stiff boulder clay	1.6	16

By the term, "puddle clay", is meant a pure, homogeneous, plastic clay, free from sand or stones. As to the time element, it is stated that all the tests were of considerable duration.

In the discussion of the paper, Dr. Unwin called attention to the important fact that Collin, in 1846, found that the resistance to shear in clay produced in 12 or 15 min. was only one-third or one-fourth of that produced in 12 to 15 sec. Mr. Bell stated that experiments were made also on perfectly dry sand, for which it was found that practically $k = 0$, and that the results were in agreement with the equation, $q = f\, p_n = \tan.\ \phi\ p_n$, where, in this case, ϕ was the "angle of repose", provided the sand was rammed in the cylinder of the testing apparatus; but if it was merely poured in and shaken, then it was found that the angle, ϕ, of the equation was much less than the angle of repose.

Enough has been given to show the absolute need of a series of comprehensive tests to determine the coefficients, f and k, for every class of material. It is highly desirable, too, to have a permanent testing laboratory, with an apparatus capable of subjecting the earth to pressures varying from 0 to at least 10 tons per sq. ft., to which earth could be sent from any locality to have its coefficients, f and k, ascertained.

* The experiments were made under the direction of Mr. A. L. Bell, the results being published in *Minutes of Proceedings*, Inst. C. E., Vol. CXCIX, Session 1914–15, Part I, in a paper by Mr. Bell entitled, "The Lateral Pressure and Resistance of Clay and the Supporting Power of Clay Foundations."

In designing the apparatus for testing the earth, it will be unfortunate if certain objectionable features of some previous designs are repeated. Thus, in Leygue's peculiar apparatus, a certain portion of the weight of earth and load was held up by the vertical sides of the box by friction, for which allowance had to be made. No error from this cause attaches to the very simple design shown in Fig. 1, where there is only contact of earth on earth and no contact of metal on metal. Likewise, in testing, although the effect of ramming and puddling should not be neglected, it seems reasonable to suppose that the most correct determinations of f and k could be obtained from the earth cut out of the bank or in its virgin state, especially when subjected to the pressure it bore in the bank. Then, too, careful experiments should be made to determine the effect of the time element, particularly for a substance like clay.

It seems needless to point out the practical importance of such experimenting, for it has long been recognized that, although certain theories of earth pressure give fairly correct results when the filling consists of a strictly granular material, such as sand, gravel, or rip-rap, yet when such filling consists of ordinary earth, endowed with cohesion, it is found that the theory is inadequate or, strictly speaking, that it is inapplicable, and that the theory for coherent earth must be applied.

Take, for example, the earth in the embankment or reservoir dam which failed, as previously cited. Possibly the fresh earth had a natural slope of 1 on 1½, before it was rolled, etc., and if placed behind a retaining wall, the thrust would have been computed (for $\phi = 33° 41'$) for an earth devoid of cohesion, on the supposition that the thrust thus found was in excess and, therefore, on the side of safety.

Suppose, however, that this filling was laid in horizontal layers and wetted and rolled or tamped, so that, as already found, the "angle of friction" (not the maximum inclination of the surface possible) was only 8° and k was 400 lb. per sq. ft.; then, perforce, the theory of coherent earth would have to be applied in order to obtain the true thrust. For a similar illustration, take the retaining walls designed by Gustav Lindenthal, M. Am. Soc. C. E., for a concrete viaduct 65 ft. high. These walls were thin and vertical, were placed on each side of the roadway, and were connected by tie-rods, the space between being filled with earth thoroughly tamped and consolidated.* In this

* *Engineering News,* May 6th, 1915.

case, the earth thrust can only be guessed at, unless the coefficients, f and k, are determined by experiment, in which case the theory of coherent earth will give the exact thrust. In fact, it is seen that Equation (1) exactly applies in finding the resistance to sliding along the plane of rupture.

Again, Mr. Bell's experiments, already cited, were made with the express purpose, not only of finding the earth thrust from the material surrounding the monoliths, but likewise the permissible pressures to which the clay under the monoliths could be subjected. The latter involved the theory of a possible heaving of the wall at the heel from the earth pressure behind it, or a possible heaving of the earth in front of the wall from the pressure of the latter at its toe. For the foundation, it was assumed, for the stiff boulder clay encountered, that $k = 1.5$ tons per sq. ft. and $\phi = 15$ degrees. The results would have been absurd, if k had been assumed equal to zero, as in the ordinary theory. In fact, this ordinary theory (for $k = 0$) is rarely ever applicable to foundations, which almost invariably are in consolidated earth or clay having a high coefficient of cohesion.

In addition to the subjects previously mentioned, may be cited the earth pressure on the bracing of trenches, the pressures on tunnel linings, and on piling, and the stability of slopes, none of which can be properly treated without experimental determinations of the coefficients, f and k, for the material encountered.

The theory of pressures in coherent earth is at hand—at least for homogeneous earth—but it cannot be applied without a more extended knowledge of the coefficients f and k. In some of the recent texts, there has been a tendency to pull down existing theory. One aim of this paper is to encourage the building up of a better and more comprehensive theory—one which deals with coherent earth for the general case, and reduces to the ordinary theory for materials nearly devoid of cohesion, such as sand, gravel, or rip-rap. Such a general theory introduces all the vital elements which are necessary for a solution.

It is fully realized that changes may occur in the values of f and k from weathering, vibrations, freezing and thawing, rains, and chemical changes, so that a prudent engineer will anticipate and allow for such changes; but this, again, only emphasizes the need of experiments on a particular earth covering a long period of years.

The term, "angle of internal friction", has been introduced into engineering literature in recent years, causing much confusion and mental perplexity; for, since for dry sand, the law, $q = f\ p_n$, practically holds, then, $f = \tan. \phi$ is the same for the interior as for the surface, and ϕ is then truly the ordinary "angle of repose". For a homogeneous, coherent earth, however, $q = f\ p_n + k$, where k and $f = \tan. \phi$ are constants, though now, ϕ no longer represents the possible maximum surface inclination, but the "angle of friction", as found from experiment, after the manner previously indicated. Thus, there should be no possible confusion of terms in introducing this new (old) theory.

The subject has been brought up at this time because the Society, through one of its Special Committees, is now engaged in a general investigation of the bearing value of soils, earth pressures, etc., and, in the writer's opinion, the most hopeful method of attack is along the lines indicated herein.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

PEARL HARBOR DRY DOCK

Discussion.*

By Messrs. Herbert E. Bellamy, B. C. Laws, and Edward Box.

Herbert E. Bellamy,† Assoc. M. Am. Soc. C. E. (by letter).—This admirable paper is most interesting, and is certain to hold the attention of a very large number of the members of the Society, as all are hoping for the successful solution of the manifold difficulties and problems which the construction of the Pearl Harbor Dry Dock has involved. Mr. Bellamy.

To the mind of the colonial engineer, the question of a dry dock of the floating type for economy naturally presents itself; but it would be useless to consider this point further, as the author specifically states:

"All questions as to the type of dock to be provided are in this case beyond discussion, inasmuch as the Congressional authorization for the work distinctly directs the construction of 'one graving dry dock'."

It is hoped, however, that the author will supply additional information on the subject that he has submitted for discussion, namely, the dredging operations, completed in 1912, connecting the harbor with the sea, as described under the heading "Location". It is stated that these operations required the removal of 4 645 000 cu. yd. of material, throughout a channel 5 miles long, at a cost of $3 296 000. It would be of very great interest to know the type, size, and number of dredges used, the method of working, delays, etc., and full particulars while the dredges were engaged over the entrance bar where there was a heavy sea swell at all times.

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* Discussion on the paper by H. R. Stanford, M. Am. Soc. C. E., continued from November, 1915, *Proceeidngs*.

† Melbourne, Victoria, Australia.

Mr. Bellamy. The progress of the works under discussion will be closely followed, especially by engineers resident in Australia, in which country extensive naval works are in contemplation.

Mr. Laws. B. C. LAWS,* ESQ. (by letter).—The writer regrets being unable to enter into a technical analysis of this interesting paper, although he has been able to peruse it sufficiently to realize the great instructive value it must have for all engineers concerned with work of this type, who may have the opportunity to read it.

The temporary setback, due to the collapse of the coffer-dam on February 17th, cannot but have had a depressing effect on those engaged in the design and construction of the dock, but such an interruption to work, unpleasant as it undoubtedly is at the time, is often a blessing in disguise; it sets us thinking, and frequently new problems or new phases of the old problem, which otherwise would have remained latent, are evolved, the solution of which has a usefulness and application which reaches beyond the work in hand.

Congratulations should be extended to the author and those associated with him, who, after careful and exhaustive analysis of all the factors in the case, have arrived at a course of procedure which bids fair to terminate with great success.

Mr. Box. EDWARD BOX,† ESQ. (by letter).—The writer has been much interested in this very lucid description of the attempted construction of a graving dock at Pearl Harbor.

Whether one regards the undertaking as a reasonable attempt to "direct the great sources of power in Nature for the use and convenience of man", or as a badly conceived scheme which disregarded the laws of Nature, one cannot but admire the very candid way in which the subject has been presented in this paper which should be of estimable value to the student of engineering, and a contribution of more than ordinary value to the world's records of engineering works.

After reading the carefully prepared description of the locality in general, coupled with the particular nature of the site, one cannot help but ask, how was the decision arrived at, which caused Congress, in May, 1908, to specify particularly a graving dock?

As a representative of Great Britain, the writer had the honor of reporting to the International Congress of Navigation, held at Philadelphia in 1912. The subject assigned to him was "Means for Docking and Repairing Vessels". As he then discussed the relative merits of the two systems of dry docks in vogue, *viz.,* graving docks and floating docks, it is not necessary at this time to repeat the views then expressed. Those who are sufficiently interested in this subject will be familiar no doubt with the papers read before that Congress. The

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* Hull, England.

† Newcastle-on-Tyne, England.

writer, however, will repeat here that it is generally recognized that there are two equally efficient methods of laying dry, for painting and repairing purposes, the larger types of ships, whether they are ships of war or of commerce, *viz.*, the graving dock and the floating dock. Mr. Box.

The site, as described in the paper, is one which seems particularly to favor the adoption of the floating dock. The author discusses the question by reference to the Congressional authorization for a graving dock, but Congress went further than that in its instructions, by limiting the cost to $2 000 000, and one becomes interested to know how that amount was arrived at, and to ask whether a graving dock would have been insisted on, had it been known that there was likely to have been so much uncertainty in the cost of its construction.

If, however, we regard the undertaking purely from the point of view of an engineering problem to be achieved at all costs, as we are asked to, the relative merits of the two kinds of docks becomes a separate question, although none the less an important one, and the only feature that remains for discussion, is how to obtain the end in view. The past history has been a chapter of failure for the reason that the difficulties of the undertaking—the writer thinks there can be no doubt about it—were not properly realized when the method of construction was decided on.

In a case, some years ago, where a project was under consideration for a Government graving dock on the Atlantic Coast, under conditions somewhat similar to those at Pearl Harbor, the writer was interested in the preparation of the estimates. The estimated cost was more than $5 000 000. The figures were based on harbor work prices, and, therefore, were probably fairly reliable. This estimate was for a dock considerably less in dimensions than that being built at Pearl Harbor. The project was abandoned, and a floating dock was constructed at much less cost.

As an engineer, the writer has long cherished the desire to build a dock on a site presenting difficulties such that the usual methods of construction would no longer suffice. He has never known a case, however—although there may have been one—such as Pearl Harbor presents, where a graving dock has been attempted. What he has had in mind is a modified design for a floating dock to be sunk on a dredged site and then converted into a concrete graving dock by sectional treatment.

To his mind, a structure complete within itself, although built first in sections, sunk into place, and then treated sectionally, is the preferable method, where practicable, of dealing with a problem such as Pearl Harbor would appear to present. In this way the question of compressibility is practically eliminated, and equal loading throughout is obtained.

Mr. Box. With regard to depositing concrete, the best concrete obtainable is that which is deposited in the dry and flooded on initial set. In graving dock work, the writer never uses concrete of less strength than $5\frac{1}{2}$ to 1 of cement, which would be considered a weak mixture in the present case. The proportion of sand to ballast, of course, depends on the grading of the particular materials used.

The site has been badly tampered with and, for this reason alone, whatever method is adopted, further difficulties must of course be expected, the worst feature of which appears to be the piling of the ground. With the necessary funds, tenacity of purpose, and under the guidance of practical engineers of matured experience, it is possible that some day the Pearl Harbor Dry Dock will be an accomplished fact.

The writer cannot close his remarks without expressing sympathy—a sentiment almost unknown with reference to engineering work—with those who have been laboring so hopelessly for years on what must have been regarded by many as an impossible task.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE ACTION OF WATER UNDER DAMS

Discussion.*

By John C. Oakes, M. Am. Soc. C. E.

John C. Oakes,† M. Am. Soc. C. E. (by letter).—This paper is of particular interest to those engaged in the design and construction of river improvement works. Mr. Oakes.

In the writer's opinion, the model used in the experiments might have been improved by increasing its size, giving greater space beneath the sheeting, and more room for the escape of tail-water. The contraction beneath the sheeting, and the short distance between the ends of the platform and box, may have had considerable effect on the resulting pressures.

As might be expected, perhaps, the data in Tables 1 and 2 (Plates XXVI and XXVII) shows some inconsistencies which are more or less puzzling. For instance, Tubes 2, 5, and 8 had their entrances at practically the same level, yet in some experiments the height of the water in Tube 8 was greater, and, again, in other experiments, much less than in Tubes 2 and 5. In Experiment No. 1, Observation No. 2, the values for Tubes 2 and 8 are, respectively, 0.50 and 0.93, yet, in Experiment No. 3, Observation No. 3, the values are 3.43 and 2.86.

Figs. 9, 10, and 11 show pressure curves for different heads and different lengths of base. For a 5-ft. head, the pressure at the heel, with bases 4.25 and 8.25 ft. long, is greater than that for the base 6.25 ft. long. There is no logical reason for such a difference, and every reason why the pressures at the heel, under the conditions shown, should have been approximately the same; if any difference was to be

* Discussion of the paper by J. B. T. Colman, Assoc. Am. Soc. C. E., continued from November, 1915, *Proceedings*.

† Major, Corps of Engrs., U. S. A., Louisville, Ky.

Mr. Oakes. expected, it would seem that the pressure should have been less for the shortest base, increasing to greatest for the longest base, on the theory that the slope of the pressure curve would be greatest when the path is shortest.

Figs. 12, 13, and 14 show pressure curves for the base 8.25 ft. long, with sheeting 1, 2, and 3 ft. long, under the heel. These curves show the smallest pressure at the heel when the shortest piling was used, and the greatest pressure when the longest piling was used. This is exactly the reverse of what should have been shown. The points plotted for Tubes 22, 26, 30, 33, and 36, for 2 and 3-ft. piling, are almost exactly in the same location for each case, and the pressures in Tube 16 alone show reduction for the 3-ft. piling, and there is an increase for Tube 46 for the same piling. By comparing Figs. 11 and 12, it will be noted that the pressures are very much reduced by using 1-ft. piling, particularly with the greater heads, and, logically, the longer piling should reduce the pressures still further.

These inconsistencies can probably be explained by the fact that sufficient time was not allowed to develop the pressures which should have been developed by the various heads recorded, and that there was some leakage through the sheeting. The author does not state how the head was held, but has mentioned that the time allowed for the head to adjust itself was not less than 2 hours. It seems probable that this time was not sufficient, otherwise it is difficult to account for the results. The author would have obtained much better and more consistent results if the head had been maintained at definite heights for 6 hours or more before readings were made.

However, the illustrations showing lines of equal pressures—Figs. 2 to 8, inclusive—are very interesting and instructive. They appear to confirm previous assumptions of the loss of head throughout the length of flow, and the effect of sheet-piling in increasing the length of the flow lines.

W. G. Bligh, M. Am. Soc. C. E., has stated* with reference to sheeting under dams, as follows:

"It has been ascertained experimentally that the value in increase of length of percolation due to a vertical obstruction is double that beneath a horizontal apron, for the reason that the percolating current travels down one side of the obstructing curtain wall and up the other."

The writer believes that this principle has been accepted, generally, for determining the length of travel for water under dams, when sheet-piles have been used. From the illustrations previously mentioned, it seems that a more conservative and correct length of flow would be obtained by taking the length of the sheeting below the river bed, plus the distance from bottom of sheeting to toe of base.

* *Engineering News*, December 29th, 1910, p. 709.

The writer cannot agree with the author's conclusions that the porosity, effective size, and uniformity coefficient have little influence on the upward pressure. Mr. Oakes.

Other writers have shown the error in the author's equations for the pressure curve and for total pressures under the dam.

The most important of the author's conclusions, and the ones to be borne in mind by designing engineers, are that sheeting at the heel of the dam reduces the pressure under the floor, and that sheeting at the toe increases it; that sheeting at the heel of a dam, to be effective, must be tight (very small leakage destroying its action); and that sheeting at the toe should be loose in order to prevent increasing the upward pressure on the floor of the structure.

Although the academic discussion of pressures under a dam on sand foundation is of interest, the rules and conclusions deduced from experiments on models must not be allowed to affect too greatly engineering practice in the designing of actual structures. The use of the model in this case introduced several conditions which do not exist under actual structures, such as practical uniformity of sand, limitation of escape of seepage by bottom, sides, and end of box, and contraction under sheeting.

Practical rather than theoretical considerations are of the greater importance, because the conditions which the experimenter is able to obtain with a model can never be obtained in the field. For instance, it is noted that the author used a pressure on the sand under the platform of from 125 to 640 lb. per sq. ft. It may be that such pressures are developed under some dams on sand foundations, but such cases will be exceptional. Very seldom, if ever, will masonry dams or weirs be designed to rest directly on sand without supporting piles. Certainly, in river work, no one would think of building such structures on sand without using both round and sheet-piles, in which case there would be practically no pressure on the sand. If the bed of the foundation is below the water level, and pumping is required, there will be a small space between the base of the masonry and the sand, caused by erosion by the flow of water from under one block of masonry while the next one is being placed. In other words, the structure is ordinarily supported wholly by the piles, with almost a certainty that there will be a space between the sand and the masonry. Under such circumstances, any upward pressure that develops under the dam will be uniform from the sheet-piling to the toe, and will depend on the tightness of the sheeting and the ease of escape below the dam, rather than on ideal laboratory conditions obtained with a model.

Another practical consideration which must not be lost sight of is the difficulty, almost amounting to an impossibility, of obtaining a tight sheeting, unless excavation is made to the full depth and an impermeable wall is constructed. The writer is firmly convinced that

Mr. Oakes.

not more than once in a hundred times will driven timber sheeting be tight. When interlocking steel piles are used, there is grave danger that the piles may be forced apart while being driven, destroying the effectiveness of the sheeting. Even when this does not occur, there will still be considerable leakage through the joints between the piles, particularly when first driven. Considering, therefore, that a tight sheeting is a very rare occurrence, and that most dams and weirs on sand will be supported on piles with a space between the masonry and the sand, full upward pressure must be assumed. Furthermore, if dams to be built on sand are properly designed for stability against sliding and overturning, and seepage is reduced sufficiently to prevent piping and blow-outs, by a proper combination of sheeting and width of base, the structures will be found, in most cases, to be capable of withstanding full upward pressure. Typical designs of certain structures, however, such as reinforced concrete dams, and the masonry bases of movable dams, will not provide sufficient weight to withstand such pressure, and the weight must be increased, or some other means, such as hold-down piles, must be used to insure stability.

Among the more important structures being built at the present time on sand foundations are the locks and dams in the lower Ohio River. These works are being built to provide a navigable depth of 9 ft. throughout the length of the Ohio. Movable dams (Chanoine) are being constructed with normal lifts of from 7 to 9 ft., and maximum lifts as great as 16 to 17 ft. The greater lift at a dam occurs when the lower pool has been lost, due to the maneuvering of the next lower dam, when there will be low water below and an upper pool above the dam under consideration. The locks are 600 by 110 ft., inside dimensions (usable). All the locks and dams below Louisville. twelve in number, must be built on sand foundations, and the means of preventing failure of these works, due to upward pressure, piping and blow-outs, and undermining by erosion below the dams, have been of the greatest interest and anxiety to those engineers who are responsible for the design and the construction of these works, which are exceptional in size of locks, lengths of dams, and heads to be withstood by river structures on sand foundations.

During the last three years the writer has had charge of the construction of the Ohio River Locks and Dams Nos. 43 and 48, both being built on sand foundations. The typical plans provide for a timber sheeting 25 ft. deep, of Wakefield piles, to be driven under the toe of the land- and guide-walls, under both toe and heel of the river wall, and under both up-stream and down-stream edges of the sills. The plans originally called for the same kind of piling under the heel of the dam. All structures are supported on round and sheet-piles.

After the difficulties of driving timber piles to form a tight sheeting had been thoroughly proved, and full consideration had been given

Mr. Oakes.

to the insecurity of structures on sand foundations, a line of interlocking steel sheet-piling, 40 ft. long, was substituted for the Wakefield piling under the dams, and a heavy lock floor, 4 ft. thick, held down by round piles, replaced a light floor, 18 in. thick, without piles. The writer has watched the construction of these two works for three years, has noted the careful efforts to produce a tight sheeting with the timber piles, and has concluded that it is almost impossible to do so. This confirms his previous experience and that of several other engineers with whom he has discussed this matter. Certainly, in the design of structures on sand foundations, the assumption cannot safely be made that a tight sheeting can be provided by driving timber sheet-piles. Better results can be obtained with steel piles, but one never knows when the sheeting is disrupted in driving, and there is always more or less leakage, even without disruption.

In the same manner the writer has watched the placing of concrete, and in no case has he found, on the works under discussion, that the concrete rested on the sand after sufficient time had been given it to set. The bed of the foundation of the various parts of these structures is about 10 ft. below low water. Construction has been carried on within coffer-dams, and during stages of the river from low water to 14 ft. above it. Owing to the permeable nature of the material, there has always been considerable percolation, which has required pumping to keep the pit sufficiently clear of water to enable construction to proceed. The water escaping from under the concrete already placed carries away the fine material directly under the concrete, leaving a space between it and the sand through which the transmission of pressure will be direct, and, consequently, any pressure which may be developed will be uniformly exerted over the whole base of the structure.

However interesting it may be to know what the upward pressure under a dam would be under ideal conditions (assumptions), such as tight sheeting, uniform material under dam, uniform weight on sand, etc., it must be borne in mind that such conditions will never be obtained throughout the length of the dam as actually constructed; that the computed pressure based on those assumptions will be the least possible pressure; and that the dam must be designed to withstand at all sections the greatest possible pressure, which will be, under ordinary circumstances, that due to the full hydrostatic head.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE ASTORIA TUNNEL UNDER THE EAST RIVER FOR GAS DISTRIBUTION IN NEW YORK CITY

Discussion.*

By Messrs. Thomas H. Wiggin and James Forgie.

Thomas H. Wiggin,† M. Am. Soc. C. E.—In the Catskill Aqueduct pressure tunnel siphon the rock conditions were similar to those described by the author. The first case was in the sinking of Shaft 4 of the Rondout inverted siphon which crosses the valley of Rondout Creek near High Falls, west of Kingston, N. Y. This was described in a paper‡ by J. P. Hogan, Jun. Am. Soc. C. E., and a discussion by the speaker adds other data, notably some from a very interesting shaft of the Detroit Salt Company. In Shaft 4 of the Rondout Siphon heavily water-bearing rock was encountered, as had been predicted from the preliminary borings and pumping tests in certain of these borings. At a depth of about 225 ft. the inward flow was more than 800 gal. per min. At first ordinary drill holes, and finally diamond-drill borings, made around the periphery of the shaft from the level to which sinking had progressed, were grouted, after which excavation was resumed with continued precautions as to drilling, carrying a pilot hole ahead and grouting through water-bearing drill holes. The leakage was very much reduced by the grouting, and would doubtless otherwise have reached very large quantities. In grouting this shaft, 971 barrels of cement were used. Some of the seams were wide, 8 in. being the maximum. Mr. Wiggin.

The grout when encountered in the seams during subsequent excavation was like moderately soft limestone, and showed very fair strength.

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* Discussion of the paper by John Vipond Davies, M. Am. Soc. C. E., continued from November, 1915, *Proceedings*.

† New York City.

‡ *Transactions*, Vol. LXXIII, 1911, p. 398.

Mr. Wiggin.

Later, in the tunnel driven from this shaft, the same wet rock strata were encountered, again in the position expected from the borings, and similar grouting was done in advance of excavation, though not so successfully. The maximum leakage into the tunnel in this stretch was about 2 000 gal. per min. This did not interfere so much with operations as it might have under other conditions, because the tunnel was driven up on a 15% grade. The water made a very pretty cascade. A heavy concrete bulkhead, with a door, had been built between this stretch of tunnel and the shaft, in order to avoid loss of the tunnel in case of meeting a sudden inflow.

Much of the rock in this piece of tunnel was divided into very small blocks by water-bearing seams, and the impression was gained, from such grouting as was done, that general impregnation by grout would be impossible with any practicable number of holes. Hence, the leakage was taken care of by a system of bottom drains and a shield of steel angle ribs and plates laid shingle-wise, which kept the water off the concrete while it was being placed. The space behind the steel was at first "dry-packed" with rock fragments, and was grouted after the concrete lining had been placed within the shield. The drains were also grouted. This method resulted in making this very wet portion of the siphon almost bone dry when completed.

In the Hudson Tunnel, where grouting pressures of about 1 000 lb. per sq. in. were used, the rock is generally solid and hard. It is what most people, from casual examination, would call a granite; geologists call it granodiorite. In general, the rock was rather tight, but within 2 or 3 ft. of a drill hole that was almost dry, another hole struck a flow of about 200 gal. per min. which temporarily overcame the pumping facilities causing the heading, which was then about 275 ft. long, to fill.

After the tunnel was pumped out, the leakage was collected in pipes, and a concrete bulkhead was placed at the end of the heading and anchored to the rock by steel rods. Four or five additional drill holes were made into the water-bearing region, which was identified by feelers as a narrow seam across the tunnel. The combined flow of all holes previous to grouting was found to be about 550 gal. per min., but was under control by gated pipes. A ground-water pressure of 400 lb. per sq. in. was recorded when the pipes were closed. The seam was finally grouted through the drill holes under a pressure of 1 000 lb. per sq. in. The tunnel at that point is more than 1 100 ft. below sea-level, and this high grouting pressure was required to overcome the ground-water head and the friction of the grout.

The equipment generally used on the Catskill works for grouting is a tank with top door and various pipes arranged so that the grout may be mixed by the release of air through the bottom pipe and ejected by forcing air into the top of the tank. This is the so-called Canniff

grouting machine. In the Hudson Tunnel, air at a pressure of 1 000 lb. was not available for forcing out the grout. At first the attempt was made to pump the grout by plunger pump but the pump valves soon wore out. The pump was then repaired and used to force water instead of air into the grouting machine on top of the grout, thus causing gradual dilution as well as displacement of the grout, but doing perfect grouting nevertheless. Mr. Wiggin.

A heavy concrete bulkhead, with a thick hinged cast-steel door, had been placed across the tunnel to prevent another flooding, and a liberal pumping equipment was put in. The tunnel was then driven through the wet ground, and a narrow seam which had been grouted could be seen across the roof and sides of the tunnel, with only a few drops of water coming from it here and there.

This Hudson Tunnel, which is about 3 020 ft. long, had been explored by about a dozen vertical holes and by two pairs of inclined borings, the longest of which was about 2 052 ft. The holes of each pair of inclined borings passed each other near the middle of the river and solid core was taken from all the holes, so that it was known that the tunnel would be in good granite nowhere less than 150 ft. thick above the tunnel. Although considerable water in the aggregate was encountered in these holes, as is usual in granite, so large a flow concentrated at one seam was surprising, particularly as at that point the rock was not less than 700 ft. thick above the tunnel. This experience serves to emphasize the fact that in many kinds of rock very wet seams may be found, surprisingly close to very dry ground and under a thick roof of apparently sound rock.

The speaker has been very much interested in Mr. Davies' paper, and particularly in the success of the method of grouting a large number of holes, and grouting back into the body of the rock. This process has been demonstrated very beautifully by this tunnel, and also by the Catskill work, and is doubtless bound to be used more and more in difficult ground. At the same time, the speaker thinks that engineers who have had experience with wet ground will be inclined more and more to keep away from it, even at a considerable expense for exploration and the deepening of the tunnel.

On the Catskill work it was found that depth was no particular drawback. When the engineers started planning pressure tunnels, 200 ft. was thought to be pretty deep for such purposes. Some of these early studies look rather absurd in the light of the finished work. The idea of liberal depth gradually gained force; and if some of the work were to be done over again, even greater depths would probably be used in places.

James Forgie,* M. Am. Soc. C. E. (by letter).—The author, in a highly creditable form, contributes a record of a most interesting and Mr. Forgie.

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* New York City.

Mr. Forgie. unique experience in subaqueous tunneling in decomposed and faulty rock. The writer was Associate Engineer on this work with Messrs. Jacobs and Davies, and will describe, in a little more detail, a few incidents of minor importance, which may interest at least a few members of the Profession.

On page 1466,* there is a description of the distortion of the cast-steel lining, caused by the flotation and the weight of the liquid cement, owing to shutting off the water by closing all the grout holes and placing end bulkheads preparatory to grouting. This distortion diminished the horizontal diameter and raised the roof of the lining, in addition to buckling at the bottom center joint. Though, as stated, the rings generally were taken down and re-erected, there was also a considerable length in which only the four bottom plates were removed, with the result that the roof came down to its proper place and the correct horizontal diameter was restored. Such grout as had been placed in the bottom was removed, and the bottom plates were replaced, restoring the section to its desired shape.

This, again, presented a jointed cast-iron shell, resting on the bottom template strips and wedged at the sides, but with an annular space over the roof between it and the concrete lining. To grout up the entire annular space without again causing distortion, it was decided to blow into the outside annular space, by grouting machine, pebbles, or round objects, which would act as strutting during grouting and later absorb the grouting in its interstices.

Shrapnel was thought of—a neutral thought, as this occurred in July, 1914—but pea gravel was obtained and blown behind the lining. However, before going far with this, and in order to prevent the roof from rising, it was decided to blow in dry cement through all the grout holes in the roof-plates. By doing this, the cement was well distributed over the roof and set by the damp from the seepage, forming a complete strutting or reaction to any tendency to rise during the final grouting.

All the grout holes and annular end bulkheads were then closed, and the grouting was completed successfully.

Fig. 23, an exaggerated sketch, shows the distortion diagrammatically, and Fig. 24, also an exaggerated sketch, shows the dry cement injected in place over the roof.

It will be understood that the bottom grout holes could not be closed before obtaining this roof reaction, and that, with the grout holes open, liquid cement would have flowed back into the tunnel.

Another minor detail of interest in connection with the accurate placing of the cast-steel lining, as described on page 1464,* was the method of placing the bolt pitch line of the four bottom plates on

* *Proceedings*, Am. Soc. C. E., for August, 1915.

Mr. Forgie.

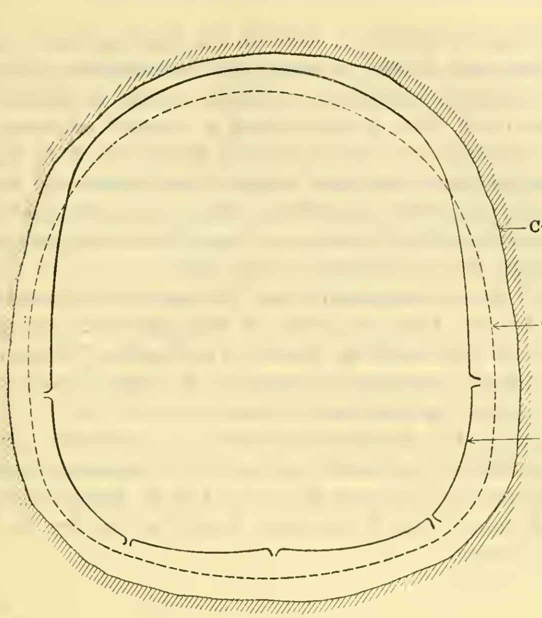

FIG. 23.

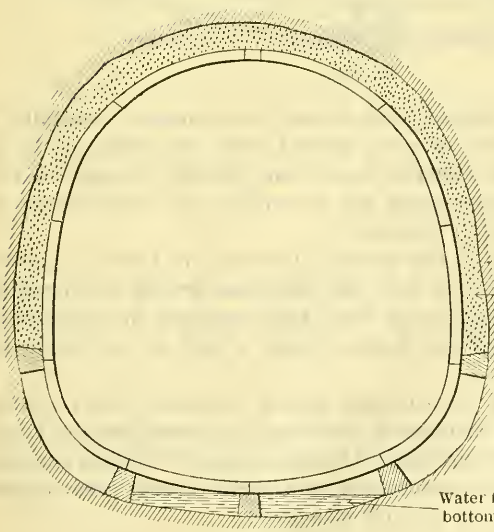

FIG. 24.

Mr. Forgie. the desired common curved plane. Though the castings were machine-faced on the joints and drilled to template, the relation of the pitch line of bolts to the back of the segment plates was, of course, variable.

To make this pitch line in the tunnel a constant distance from the accurately laid strips of iron on which the plates were imposed, set-pins were tapped on to the back edges of the plates and cut to a length which made their ends a uniform distance from the pitch line.

This work was done in the yard and saved much expensive field work in the tunnel. It is illustrated by Fig. 25.

On page 1434* there is a description of the method of consolidating the joint made by the fresh concrete of the side-walls and the set concrete of the arch previously in place. The wooden chutes carried up above the joint also performed somewhat the same function as a "sinking head" does in the pouring of a casting.

Mr. Wegmann, if the writer understands his remarks, seems to regret not having carried out, under contract by a contractor, a similar piece of work under the Harlem River at 155 ft. below mean high water, instead of doing it as it was done finally in the dry at 307 ft. below mean high water.

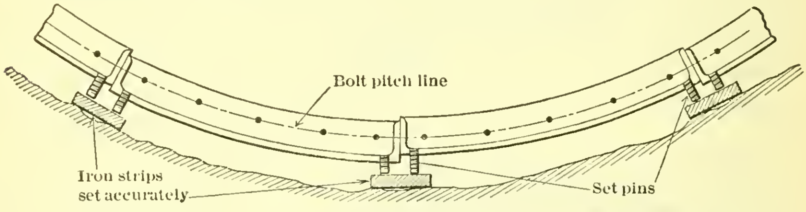

FIG. 25.

In the first place, it would have broken the contract; secondly, a contractor, in the writer's opinion, should only be called on to do such work on a time and material basis; and, thirdly, though, in this Astoria case, the experience could not be avoided, it should under no circumstances be deliberately planned.

However, in the case of the Astoria Tunnel, the lines of defense were always arranged so that, had the bad zone proved impregnable, the remainder of the tunnel could have been retained in conjunction with a deep by-pass under this fault at such a level as to obtain dry work.

Ten years ago when considering boring through faulty water-bearing rock, Mr. Jacobs instructed the writer to investigate the Kind-Chandron method of shaft-sinking, a German means of infinite patience by which shafts of circular cast tubing were sunk to very great depths against enormous water and ground pressures.

* *Proceedings*, Am. Soc. C. E., for August, 1915.

The writer inspected this method as used to a depth of about 1 200 ft. at the Kent Collieries, near Dover, England, where it passed through water-bearing calcareous grit, marl, limestone, sand, sandstone, and clay. Mr. Forgie.

Its use is dependent on enormous percussive pulverizing force, and, of course, in a vertical direction. Without detailing the process, the writer thinks that it forms at least a nucleus for a method of carrying out such work horizontally, as in a tunnel, by other means of pulverization than the force of gravity; in brief, such an arrangement would form a trepan-shield with a closed diaphragm of hydraulic dimensions, balanced, as far as possible, by permitting the water to pass through, and carry the pulverized material with it. This Kind-Chandron method of shaft-sinking (vertical tunneling) is the only successful method for rock-boring under such conditions, which the writer has noted thus far, and, as its progress in the instance described was only a few inches per day, the need for its use has to be a very special one.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

INDUCED CURRENTS OF FLUIDS

Discussion.*

By John C. Trautwine, Jr., Assoc. Am. Soc. C. E.

Mr. Trautwine.

John C. Trautwine, Jr.,† Assoc. Am. Soc. C. E.—This paper reminds the speaker that, in certain experiments, where water entered and left a cylindrical box tangentially, the placing of a wire-cloth screen in the box, instead of increasing the loss of head, as expected, diminished that loss, evidently by diminishing or destroying the centrifugal force, due to the previously unchecked rotary motion of the water.

The speaker is reminded also that Messrs. Gaskell S. Jacobs and F. A. Sooy have described‡ a simple method, devised by Mr. Buckner Speed, of measuring the velocity of water flowing through a pipe in which a bend occurs. This consists simply in tapping the bend on its outer and inner sides. Then, if h equals the difference in pressure, in feet, between the outside and the inside of the bend, R equals the radius, in feet, of the bend described by the pipe axis, and d equals the pipe bore, in feet; we have, for the velocity, in feet per second,

$$v = c \sqrt[n]{h \frac{R}{d}}.$$

According to Messrs. Jacobs and Sooy, $c = 5.6$, and $n = 1.9$.

The author appears to maintain that the resistance due to a bend is proportional to the sharpness of the bend. Messrs. Williams, Hubbell and Fenkell, found§ that, in a given length of pipe, with given total deflection, the resistance is practically the same, whether the entire pipe forms a single easy curve, or whether there is only a single short sharp curve, the remainder of the pipe being straight.

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* Discussion of the paper by F. zur Nedden, Esq., continued from the November, 1915, *Proceedings*.

† Philadelphia, Pa.

‡ *Journal of Electricity, Power and Gas*, San Francisco, July 22d, 1911.

§ *Transactions*, Am. Soc. C. E., Vol. XLVII, April, 1902.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE HYDRAULIC JUMP, IN OPEN-CHANNEL FLOW AT HIGH VELOCITY

Discussion.*

By Messrs. Carl B. Andrews, R. D. Johnson, Mansfield Merriman, E. G. Walker, and Edward W. Bush.

Carl B. Andrews,† Assoc. M. Am. Soc. C. E. (by letter).—In the author's discussion of the depth of pool necessary to absorb the energy of a spillway discharge, he mentions Professor Merriman's original formula for height of jump. It would perhaps be only fair to Professor Merriman to quote the paragraph in connection with the development of this formula. It is as follows: Mr. Andrews.

"To determine the height of the jump, let $d_2 - d_1$ be represented by j. It is then to be observed that the lost velocity-head is $(v_1^2 - v_2^2) \div 2\,g$, and that this is lost in two ways, first by the impact due to the expansion of section $\left[h' = \frac{(v_1 - v_2)^2}{2\,g}\right]$, and second by the uplifting of the whole quantity of water through the height, $\frac{1}{2}(d_2 - d_1)$, loss in friction between d_1 and d_2 being neglected.

Hence $$\frac{v_1^2 - v_2^2}{2\,g} = \frac{(v_1 - v_2)^2}{2\,g} + \frac{j}{2}.$$

Inserting in this the value of v_2, found from the relation $v_2\,(d_1 + j) = v_1\,d_1$, and solving for j, gives

$$j = - d_1 + 2\sqrt{d_1 \frac{v_1^2}{2\,g}}."$$

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* Discussion of the paper by Karl R. Kennison, Assoc. M. Am. Soc. C. E., continued from November, 1915, *Proceedings*.

† Honolulu, Hawaii.

Mr. Andrews.

The subject of the height of jump has been treated by at least two other eminent engineers, Professor Hubert Engels, and Professor W. C. Unwin. Professor Engels, in his recently published "Handbuch des Wasserbaues," deduces the approximate height of the jump, as follows: The height of the jump may be obtained approximately with the aid of the law of kinetic energy, by equating the difference between the two velocity heads to the difference between the two water surfaces, or (in Fig. 18),

$$\frac{v_2^2}{2g} - \frac{v_1^2}{2g} = d_1 - d_2$$

or, since
$$v_2 = v_1 \frac{d_1}{d_2},$$

and letting
$$\frac{v_1}{2g} = k.$$

$$\frac{1}{2g}\left[v_1^2 \frac{d_1^2}{d_2^2} - v_1^2\right] = \frac{v_1^2}{2g}\left[\frac{d_1^2 - d_2^2}{d_2^2}\right] = d_1 - d_2$$

or
$$\frac{v_1^2}{2g}\left[\frac{d_1 + d_2}{d_2^2}\right] = 1$$

or
$$k\left[\frac{d_1 + d_2}{d_2^2}\right] = 1$$

whence,
$$d_2^2 - k\,d_2 = k\,d_1$$

$$d_2 = \frac{k}{2} + \sqrt{k\,d_1 + \frac{k^2}{4}} = \frac{v_1^2}{4g} + \sqrt{\frac{v_1^2}{2g} d_1 + \left(\frac{v_1^2}{4g}\right)^2}$$

or
$$j = -d_1 + \frac{v_1^2}{4g} + \sqrt{\frac{v_1^2}{2g} d_1 + \left(\frac{v_1^2}{4g}\right)^2}.$$

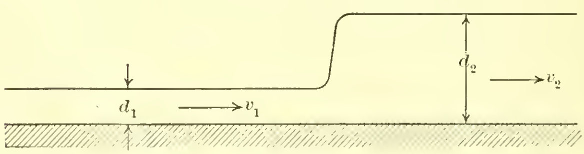

FIG. 18.

Professor Unwin's treatment of the problem appears in the *Encyclopedia Britannica,* under the head of "Hydraulics". He writes:

"Let the figure [Fig. 19] represent the longitudinal section of a stream and *ab, cd,* cross-sections normal to the bed, which for the short distance considered may be assumed horizontal. Suppose the mass of water *abcd* to come to *a'b'c'd'* in a short time t; and let u_0, u_1, be the velocities at *ab* and *cd*, Ω_0 and Ω_1 the areas of the cross-sections. The force causing change of momentum in the mass *abcd* estimated horizontally is simply the difference of the pressures on *ab* and *cd*. Putting h_0, h_1, for the depths of the centers of gravity of

ab and *cd* measured down from the free water surface, the force is $G\,(h_0\,\Omega_0 - h_1\,\Omega_1)$ pounds, [G = weight per cubic unit of liquid] and the impulse in t seconds is $G\,(h_0\,\Omega_0 - h_1\,\Omega_1)\,t$ second-pounds. The horizontal change of momentum is the difference of the momenta of $cdc'd'$ and $aba'b'$: that is, Mr. Andrews.

$$\frac{G}{g}\,(\Omega_1\,u_1^2 - \Omega_0\,u_0^2)\,t.$$

Hence, equating impulse and change of momentum,

$$G\,(h_0\,\Omega_0 - h_1\,\Omega_1)\,t = \frac{G}{g}\,(\Omega_1\,u_1^2 - \Omega_0\,u_0^2)\;;$$

Hence $$h_0\,\Omega_0 - h_1\,\Omega_1 = \frac{1}{g}\,(\Omega_1\,u_1^2 - \Omega_0\,u_0^2).$$

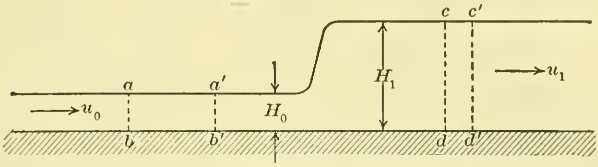

FIG. 19.

"For simplicity let the section be rectangular, of breadth B and depths H_0 and H_1 at the two cross-sections considered; then $h_0 = \frac{1}{2}\,H_0$, and $h_1 = \frac{1}{2}\,H_1$. Hence

$$H_0^2 - H_1^2 = \frac{2}{g}\,(H_1\,u_1^2 - H_0\,u_0^2).$$

$\frac{1}{2}\,(H_0^2 - H_1^2) = \frac{1}{g}\,[H_1\,u_1^2 - H_0\,u_0^2]$. But, since $\Omega_0\;u_0 = \Omega_1\,u_1$, we have $u_1^2 = u_0^2\,\frac{H_0^2}{H_1^2}$ and $H_0^2 - H_1^2 = \frac{2\,u_0^2}{g}\left[\frac{H_0^2}{H_1} - H_0\right]$

*   *   *   *   *   *   *

"Dividing by $H_0 - H_1$, the equation becomes

$$\frac{H_1}{H_0}\,(H_0 + H_1) = \frac{2\,u_0^2}{g}\,;$$

therefore

$$H_1 = \sqrt{\frac{2\,u_0^2\,H_0}{g} + \frac{H_0^2}{4}} - \frac{H_0}{2}."$$

The last equation, as printed in the *Encyclopedia Britannica*, contains a typographical error, the sign of the term, $\frac{H_0}{2}$, being there printed $+$.

A comparison of the Merriman, Engels, and Unwin formulas as given herein, will show that they are essentially different, and will not give the same result for any given set of data.

Mr. Andrews.

The writer hesitates to make what might be considered a criticism of the work of the eminent engineers whose names accompany the foregoing formulas, and what follows has been written in an endeavor to find the reason for the differences between them.

Referring to the development of the Engels formula, which he refers to as an approximate one, it seems as though the difference between the velocity heads should be equal to the difference between the heights of the centers of gravity of the water masses, in the two phases of flow, rather than equal to the difference of elevation of the two water surfaces. Thus, we should have $\frac{v_1^2 - v_2^2}{2\,g} = \frac{d_2 - d_1}{2} = \frac{j}{2}$.

This expression is also obtained by equating the loss of kinetic energy in any water mass, M, which is $\frac{1}{2} M (v_1^2 - v_2^2) = \frac{W}{2\,g} (v_1^2 - v_2^2)$, to the work done in raising the same mass through the height, $\frac{1}{2} (d_2 - d_1)$, which is $\frac{W}{2} (d_2 - d_1)$. It is also obtained from Professor Merriman's formula by omitting the term which expresses the head lost in expansion.

If this expression is used, the formula for height of jump will be:

$$j = d_2 - d_1 = \frac{v_1^2}{2\,g} + \sqrt{\frac{v_1^2}{g}\,d_1 + \left(\frac{v_1^2}{2\,g}\right)^2} - d_1 \ldots\ldots (A)$$

It appears as though Professor Unwin, in the development of his formula, has treated static pressure as if it were a dynamic force. He takes the difference of static pressures on planes, *ab* and *cd*, multiplies it by the time, t, calls the product an impulse, and equates it to change of momentum.

The expression of this operation is $Ft = M(v_1 - v_0)$, in which F must be a dynamic force which causes the change of momentum from Mv_0 to Mv_1. To put the equation into what is perhaps a more common form, we may multiply both members by $\frac{v_1 + v_2}{0}$, which will change it to a work equation. Thus, $Ft \left(\frac{v_1 + v_0}{2}\right) = \frac{M}{2} (v_1^2 - v_0^2)$, and in this, $\frac{v_1 + v_0}{2}\,t$ represents the distance through which the force, F, acts on the mass, M, and $\frac{M}{2} (v_1^2 - v_0^2)$ represents the difference of kinetic energies possessed by the mass, M, after and before the action of the force, F. In the case of the flow of water in a conduit, it does not seem reasonable to think of a difference between two static pressures acting on the fluid through any particular distance, and thus doing work on the

water, any more than differences of static pressures on moving water in a pipe might be considered to be doing work on the water as it flows. Mr. Andrews.

Table 3 contains the data of Bidone's experiments, as given by Professor Merriman, together with values of j computed by the various formulas.

TABLE 3.—DATA OF BIDONE'S EXPERIMENTS, AS GIVEN BY PROFESSOR MERRIMAN.

d_1.	v_1.	Observed j.	j computed by Merriman's formula.	j computed by Engel's formula.	j computed by Unwin's formula.	j computed by Equation (A).
0.149	4.59	0.274	0.290	0.288	0.224	0.632
0.154	4.47	0.267	0.283	0.270	0.213	0.595
0.208	5.59	0.305	0.428	0.435	0.329	0.940
0.246	6.28	0.493	0.531	0.556	0.417	1.190

It is quite evident that Equation (A), which assumes no loss of head because of eddies, does not give correct values of j, and that it should contain a term which will express such loss of head. The term inserted by Professor Merriman was of the form used in the case of sudden expansion of section of pipe: whether this or some other expression should be used may probably not be decided from theoretical considerations alone. It is conceivable that some forms of the wave may result in very little disturbance by eddies, just as some forms of pipe enlargement produce very little loss of head, and in these cases the jump should be higher than in those cases where many eddies are produced.

R. D. JOHNSON,* ESQ.—The speaker has been much interested in Mr. Kennison's able and valuable paper, and is glad to take part in the discussion. Mr. Johnson.

Mr. Kennison calls attention, in a novel way, to a well-known phenomenon of Nature. His treatment of the subject is conclusive, so far as it goes, and leaves little or nothing to be said in criticism. The speaker has always regarded these "alternative" or complementary levels in a different way from that presented by Mr. Kennison, and it may be worth while to view the matter in a somewhat more commonplace light than through the medium of a cubic equation, two of the roots of which not only demonstrate the existence of "alternative" stages, but also determine them in relation to the quantity of water flowing and the fall of potential from the static level.

A casual reading of the paper might lead one to conclude that the depth of water in a flume is an unstable affair, and that, with slight provocation, it may flop to its "alternative" and continue indefinitely

* New York City.

Mr. Johnson. to flow at a different depth. This is a correct theoretical conclusion, neglecting friction, but it might be interesting to show, in a rough way, how much it is affected ordinarily by a consideration of "slope". It may be interesting, also, to point out the simple relation of one "alternative" stage to the other, which does not involve the cubic equation, although no other method can express the whole story so completely in one equation.

For the sake of this argument, let us deal, in imagination, with a flume so wide that d and r may be regarded as interchangeable quantities, and, merely for the sake of simplicity in illustration, let C, in the old Chezy formula, be considered constant through a reasonable range of conditions. Let the flume have a smooth bell-mouth entrance in which friction may be neglected. Let the water be regarded as flowing at a uniform depth, d_1, and with a velocity, v_1, the slope of the flume being s_1. Now, at some point well along in the flume, let the slope suddenly change to a greater one, s_2. This condition would naturally cause an acceleration of the water, and if the relation of s_1 to s_2 were just right, and a suitable submerged weir were placed at the transition point, the depth, d_1, on the slope, s_1, would not be altered, but the water would "jump" or suddenly slide into another régime of flow on the slope, s_2, at another uniform, though shallower, depth, d_2, and at higher velocity, v_2. This relation of the two depths is the one so completely expressed by Mr. Kennison in his cubic equation. It may be shown that this exact condition, as described, can occur only when the two slopes bear a definite relation to each other, more or less independent of the quantity, Q, and when they are also inversely proportional to the cube of the respective depths, or directly proportional to the cube of the respective velocities. Also, that s_1 is less than $\frac{g}{C^2}$, and s_2 is greater than $\frac{g}{C^2}$, and that when each is made equal to $\frac{g}{C^2}$, and the submerged weir is finally removed, the flume will be carrying all the water that can possibly enter it, and the depth, at this time being equal to $\frac{v^2}{g}$, has no "alternative" and, therefore, cannot be made to "jump" to another value.

The relation of the velocities and depths on either side of the transition point, previously referred to, is expressed simply as follows (neglecting local eddy losses):

$$\frac{v_2^2 - v_1^2}{2\,g} = d_1 - d_2\text{; also, } v_1\,d_1 = v_2\,d_2 = Q \text{ per unit width.}$$

From these equations, we have,

$$v_1^2\,(d_2 + d_1) = 2\,g\,d_2^2 \quad\text{.......... (1)}$$

and,

$$v_2^2\,(d_2 + d_1) = 2\,g\,d_1^2 \quad\text{.......... (2)}$$

Mr. Johnson.

or, $$d_1 = \frac{v_2^2}{4\,g}\left\{1 + \sqrt{1 + \frac{8\,g\,d_2}{v_2^2}}\right\} \quad\ldots\ldots\ldots\ldots\ldots(3)$$

and, $$d_2 = \frac{v_1^2}{4\,g}\left\{1 + \sqrt{1 + \frac{8\,g\,d_1}{v_1^2}}\right\} \quad\ldots\ldots\ldots\ldots\ldots(4)$$

These equations express the relation of d_1 to d_2 for a given quantity of water ($d_2\,v_2$ or $d_1\,v_1$) flowing, and may be written equally well in terms of Q, eliminating v_1 and v_2.

The Chezy formula gives:

$$v_1^2 = C^2\,d_1\,s_1, \text{ and } v_2^2 = C^2\,d_2\,s_2.$$

Dividing one by the other, we have,

$$\frac{s_1}{s_2} = \frac{v_1^2\,d_2}{v_2^2\,d_1} = \frac{v_1^3}{v_2^3} = \frac{d_2^3}{d_1^3},$$

which expresses the necessary definite relation of the two slopes, in order that the quantity of water flowing at uniform depths may be the same in both cases. The relation of one slope to the other, necessary to maintain, indefinitely, the "alternative" water levels, may be obtained by combining these equations with Equations (3) and (4), eliminating the v and d, and becomes $\left(\frac{g}{C^2} \text{ being put} = s_m\right)$:

$$s_1 = \frac{1}{s_2^2}\left\{\frac{4\,s_m}{1 + (1 + 8\,s_m \div s_2)^{\frac{1}{2}}}\right\}^3 \text{ less than or equal to } s_m,$$

and, $$s_2 = \frac{1}{s_1^2}\left\{\frac{4\,s_m}{1 + (1 + 8\,s_m \div s_1)^{\frac{1}{2}}}\right\}^3 \text{ greater than or equal to } s_m.$$

These particular slopes might appropriately be termed "alternative" slopes.

By inspection of Equation (1), it may be seen what takes place when s_1 is increased and s_2 is decreased, bringing them together and drowning out the "jump"; for then d_1 and d_2 become equal, and we have, calling the new common velocity, v_m, and the depth, d_m,

$$v_m^2 \times (2\,d_m) = 2\,g\,d_m^2, \text{ or } d_m = \frac{v_m^2}{g}, \text{ and } s_m = \frac{g}{C^2},$$

which might be termed the "natural slope" of a flume, because it represents the minimum slope which will carry away, at uniform depth, all the water which can be accelerated into it or be made to enter it. The depth of the water is twice the velocity head, and, therefore, the surface at the flume entrance drops one-half the depth in order to accelerate the water. As Mr. Kennison has pointed out, this condition obtains when the flume offers no restriction to the flow, and the maximum possible quantity is entering it; or, in this case,

Mr. Johnson.

the point of control caused by the submerged weir has been entirely removed, and the slope evened up uniformly throughout its length.

If the distance from the still-water surface to the bottom of the flume, at the entrance, be called H, then $d_m = \frac{2}{3} H$, and $v_m = \sqrt{2\, g\, \frac{H}{3}}$, and $Q = d_m\, v_m = \frac{2\, H}{3} \sqrt{\frac{2}{3}\, g\, H} = 3.09\, H^{\frac{3}{2}}$, which is the natural formula for a smooth submerged weir, and also represents the quantity which requires a slope of $\frac{g}{C^2}$ to keep it moving at uniform depth, and expresses a stable condition of flow which has no "alternative".

The foregoing analysis falls far short of a rigid and logical mathematical discussion, and is intended only to point out some interesting physical relations which may furnish a sidelight on Mr. Kennison's work, even if it adds nothing to it.

The fact that C is not constant and that r is not equal to d will modify, of course, the numerical results derived for any particular case.

Mr. Merriman.

MANSFIELD MERRIMAN,* M. AM. SOC. C. E.—Fig. 11 of Mr. Kennison's interesting paper appears to indicate that the hydraulic jump is caused by a slight obstruction or irregularity in the bed of the channel just below the toe of the jump; this figure also shows an upward spurt of the water at the top of the jump. The speaker, having seen jumps formed in a rectangular trough, is able to affirm that they may occur without any obstruction near the toe, and that usually there is no upward spurt at the top. Undoubtedly, a so-called jump may occur when a considerable obstruction exists in the bed of the channel and perhaps such was in the mind of the author when he prepared Fig. 11. The true hydraulic jump, however, is of a different character.

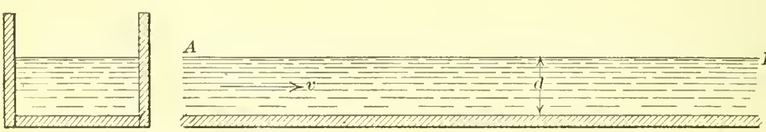

FIG. 20.

The theory of the non-uniform flow of water applied to a case like that of Fig. 20, where a rectangular trough carries water on a uniform slope, with the constant depth, d, and the constant mean velocity, v, shows that if a gate or dam be inserted at E, a change in the surface, $A\,E$, will take place in one of two ways. Fig. 21 shows the way which occurs when v is less than $\sqrt{g\, d}$, where g is the acceleration of gravity. Here the water surface is convex to the bed of the channel and extends a long distance up stream. This is the common case of backwater

* New York City.

Mr. Merriman.

which occurs in streams when the original surface of the water is raised by the construction of a dam or bridge piers.

Fig. 22 shows the second case, where the mean velocity, v, is greater than $\sqrt{g\,d}$, and here the true hydraulic jump is formed. From A to B the water is flowing with the uniform depth, d, at B the depth suddenly increases and shortly after reaches the greater depth, D, the surface of the water from B to E being concave to the bed of the channel. The distance, $E\,B$, from the dam back to the jump depends on the velocity, v, only; when v is only slightly greater than $\sqrt{g\,d}$, the distance, $E\,B$, is large; when v is very much greater than $\sqrt{g\,d}$, the distance, $E\,B$, may be quite small; but, wherever the jump may be formed, the depth, D, is theoretically always the same for the same velocity, v, and the same depth, d.

$v < \sqrt{gd}$

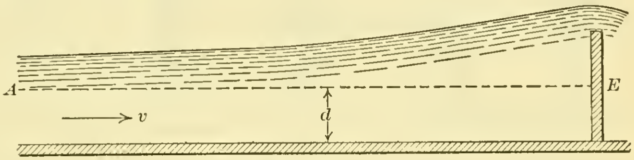

FIG. 21.

$v > \sqrt{gd}$

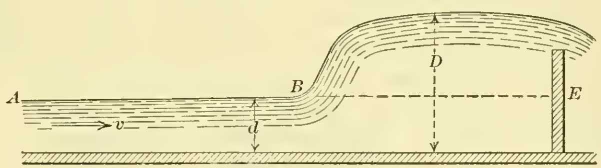

FIG. 22.

Bidone, in 1818, made the first observations on the height of the hydraulic jump, his rectangular trough being 1.066 ft. wide.* His measures were made in old Paris *pieds* and *pouces*, but they have been transformed into English feet by the speaker, and are given in Columns (2), (3), and (4) of Table 4. In Column (5) are given values of the ratio, $\frac{v}{\sqrt{g\,d}}$, as computed by the speaker, and in Column (6) are values of the lost head, computed from the expression,

$$h' = \frac{v^2 - V^2}{2\,g} + d - D,$$

in which V, the velocity corresponding to D, is found from $V = \frac{v\,d}{D}$, while g has been taken as 32.19 ft. per sec. per sec., this being the value for Paris, where the observations of Bidone were made.

* *Transactions,* Royal Soc. of Turin, 1819, pp. 21–80.

Mr. Merriman.

TABLE 4.—BIDONE'S OBSERVATIONS ON HYDRAULIC JUMPS.

(1) No. of experiment.	(2) Observed mean velocity, v, in feet per second.	(3) Observed mean depth, d, in feet.	(4) Observed mean depth, D, in feet.	(5) Computed ratio, $\frac{v}{\sqrt{g\,d}}$.	(6) Computed loss of head, h', in feet.
I_1	4.47	0.155	0.423	2.0	0.001
I_2	4.45	0.155	0.437	2.0	— 0.013
I_3	4.41	0.156	0.430	2.0	— 0.012
I_4	4.52	0.152	0.436	2.0	— 0.005
II_1	5.59	0.208	0.613	2.2	0.024
II_2	5.56	0.210	0.620	2.1	0.015
II_3	5.52	0.211	0.630	2.1	0.002
II_4	5.49	0.212	0.642	2.1	— 0.013
II_5	5.67	0.206	0.647	2.2	0.007
III_1	6.29	0.246	0.738	2.2	0.054
III_2	6.36	0.244	0.745	2.3	0.059
III_3	6.39	0.242	0.764	2.3	0.049
IV_1	4.55	0.150	0.398	2.1	0.028
IV_2	4.57	0.150	0.405	2.1	0.040
IV_3	4.57	0.150	0.428	2.1	0.007
IV_4	4.59	0.149	0.423	2.1	0.013

In 1894, a number of observations were made at Lehigh University, by Mr. Robert Ferriday, in the preparation of a graduating thesis. An abstract of this thesis was published,* but, as it gives no results of the observations, it seems best to put them on record here, since the work was done under the writer's supervision. Mr. Ferriday's trough was 0.66 ft. wide and was laid on various slopes so as to produce different velocities. The depth in each observation was determined from the mean of three measures, and the velocities were found by measuring the water caught in a barrel in a given time. Dams of different heights were inserted at the foot of the trough (E in Fig. 22), which caused the jump to occur at different distances up stream; but, for any given velocity, the height of the jump was found to be closely the same. Altogether forty-six observations were made at eleven different velocities. The results of the eleven series are given in Table 5, in Columns (3), (4), and (5); and Columns (6) and (7) give the computed values of the ratio, $\frac{v}{\sqrt{g\,d}}$, and the head, h', lost in making the jump. In these computations, the value of g for Lehigh University has been taken as 32.16 ft. per sec. per sec.

These observations on the height of the hydraulic jump are all that are known to the speaker, except those made by Darcy and Bazin in 1856.† They give several series of observations made in a rectangular timber conduit, 1.99 m. (6.53 ft.) wide. In the series

* *Engineering News*, July 11th, 1895, Vol. 34, p. 28.

† "Recherchés Hydrauliques" (Paris, 1865), pp. 284–292.

numbered 89, 90, and 91, the jump was quite ill-defined on account of the slight excess of v over $\sqrt{g\ d}$, the ratio of the former to the latter being from 1.1 to 1.3. For Series 92, the ratio, $\frac{v}{\sqrt{g\ d}}$, was between 1.5 and 1.9, and several characteristic jumps were obtained. All observations made by Bidone, however, have values of this ratio ranging from 2.0 to 2.3, and in four of Ferriday's observations the ratios are from 3.0 to 3.5. It appears unnecessary to give the observations of Darcy and Bazin here, as they may be readily consulted in their book. The speaker is inclined to the opinion that good characteristic jumps will rarely be formed unless the velocity, v, is greater than 2.0 $\sqrt{g\ d}$. The great variation in the values of the lost head given in the last columns of Tables 4 and 5, shows how difficult it is to make precise measurements on the height of the hydraulic jump; five of these values are negative, which indicates that the corresponding observations are probably of little value. Mr. Merriman.

TABLE 5.—FERRIDAY'S OBSERVATIONS ON HYDRAULIC JUMPS.

(1) No. of series.	(2) No. of observations in series.	(3) Observed mean velocity, v, in feet per second.	(4) Observed depth, d, in feet.	(5) Observed depth, D, in feet.	(6) Computed ratio, $\frac{v}{\sqrt{g\ d}}$.	(7) Computed loss of head, h', in feet.
I	3	2.18	0.050	0.143	1.3	—0.028
II	5	2.98	0.044	0.150	2.1	+0.020
III	6	3.56	0.036	0.153	3.1	0.069
IV	5	3.66	0.033	0.168	3.5	0.065
V	3	4.39	0.095	0.267	1.4	0.090
VI	4	5.02	0.083	0.285	1.9	0.155
VII	3	5.02	0.071	0.290	2.3	0.151
IX	4	3.50	0.055	0.162	2.0	0.061
X	4	3.95	0.046	0.175	2.7	0.097
XI	5	4.06	0.042	0.188	3.0	0.097
XII	4	4.33	0.038	0.205	3.5	0.114

Mr. Ferriday also noticed the fact that at the toe of the jump there was no horizontal velocity along the bottom of the trough, as balls of putty placed in the trough at the upper end moved along the bottom until they reached a point just below the toe of the jump, where they stopped. This indicates that probably the lower strata of water turn upward in order to make the jump; if so, it may be inferred that a jump formed near the foot of an apron of an ogee dam will be very advantageous in preventing scour.

Two formulas for the height of the jump are given by Bidone in his paper of 1819 and four or five others have been derived by later writers. All these formulas are imperfect on account of the neglect of the fall between the sections at d and D and other ele-

Mr. Merriman.

ments difficult to include, and also, in some cases, on account of defective reasoning. The formula of Belanger, later adopted by Rühlmann and by Weisbach, assumes that no loss of head occurs in forming the jump, and hence values computed from it are considerably in excess of observed values. Nearly all formulas, in fact, give computed values higher than those observed. To derive a reliable formula for the height, D, from observed values of d and v, it seems necessary that coefficients should be introduced, which are derived from the study of observations. No doubt this will be done in time, and, perhaps, the observations previously given may aid in deriving such coefficients.

The author's use of the term "hydraulic gradient" seems to be unfortunate, and does not agree with that generally used. When a number of piezometers are placed on a pipe, the line connecting the water levels in them is the hydraulic gradient; when water flows in an open channel, its surface is the hydraulic gradient. The term "energy gradient" better applies to the curve formed by successive values of the author's H, as this is the sum of the pressure head and the velocity head.

The author deserves much credit for preparing this interesting paper, as it deals with a subject which has been rarely discussed. Observations like those made by Mr. Ferriday, in 1894, can easily be conducted in any hydraulic laboratory, and it is hoped that further experimental work may be done in this direction. One who has seen the true hydraulic jump, under the conditions shown in Fig. 22, can never forget it, as it violates the impressions which he has formed by observing the usual backwater case of Fig. 21. In all hydraulic laboratories for the instruction of students it would be well to have the hydraulic jump exhibited occasionally.

Mr. Walker.

E. G. Walker,* Assoc. M. Am. Soc. C. E. (by letter).—The problem of the "hydraulic jump", or, as it is frequently named, the "standing wave", is one which has been before irrigation engineers for many years. The phenomena accompanying the formation of a standing wave were investigated by the engineers of the Government of India in the early days of river control and irrigation works in that country. As the author states, one may look in vain in the generality of textbooks on hydraulics for a proper discussion of the subject, but the writer wishes to point out one important exception which, apparently, has not been brought to Mr. Kennison's notice. In his article on "Hydraulics" in the Ninth Edition of the Encyclopædia Britannica, Dr. W. C. Unwin, at that time Professor of Hydraulic and Mechanical Engineering at the Royal Indian Engineering College, Cooper's Hill, developed the theory of the standing wave on a rational basis, and

* Hull, England.

worked out the conditions under which the phenomenon becomes possible.* This is the most complete exposition of the subject known to the writer. The article has been revised for the present (Eleventh) edition of the Encyclopædia. Mr. Walker.

The author, in Section 2 of the paper, starts off his theory with a postulate that "there are certain controlling sections" at which "for the given head and channel depth, the discharge is a maximum." Some further explanation of this statement appears to be desirable, for as soon as a definite régime of flow exists in a channel, no matter how complex the conditions of flow may be, the discharge at all sections must be the same, or there would be discontinuity of flow at some point. Would it not be a truer statement of the case to say that there exists a "critical stage" in the flow, dependent on the friction, form and slope of the channel bed, and the velocity of the stream, at which the water level will change from the lower to the higher?

The statement in the last paragraph of Section 2 is important. Considerable misconception has arisen owing to the incorrect application of formulas for the loss of head by sudden expansion to conditions to which they are not applicable. The sudden transition from one state of equilibrium to the other, found in the hydraulic jump, involves no energy loss, except such as is due to the fact that water is not exactly the theoretical fluid it is assumed to be. There is no reason for assuming, without proof, that results applicable to flow in pipes should be applied without change to open-channel flow under atmospheric pressure. It has been shown experimentally that such assumption is incorrect.

Although the author discredits the "sudden expansion" theory as originally propounded, he does not offer any modification to replace it. On the principle so often used in hydraulic calculations, namely, that force exerted is equal to the rate of change of momentum, Dr. Unwin develops, in the article previously mentioned, a formula for the depth of water after the jump, which is free from incorrect assumptions. His formula, in the notation of the paper, is

$$d_2 = \sqrt{\left\{\frac{2\,V_1^2}{g}\,d_1 + \frac{1}{4}\,d_1^2\right\}} - \frac{1}{2}\,d_1 \ldots\ldots\ldots\ldots (1)$$

as compared with,

$$d_2 = \sqrt{\frac{2\,V_1^2}{g}\,d_1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

the formula of the "sudden expansion" theory.

The subject has recently been investigated experimentally by Professor A. H. Gibson, of Dundee, Scotland. His experiments are described in a paper entitled "The Formation of Standing Waves in

* Vol. XII, pp. 499–502.

Mr. Walker. an Open Stream."* He applies Equations (1) and (2) to his experimental results, and also to a series of experiments of Messrs. Darcy and Bazin, and shows that, for his own experiments, whereas Equation (2) gives discrepancies of from 3 to 23% excess in the values of d_2, the range of variation using Equation (1) is only from — 4½ to + 3½ per cent. The results in the case of the Darcy and Bazin experiments are more startling, the differences being, for Equation (2), from 22 to 77%, and for Equation (1) from — 3 to + 3 per cent. By allowing for the variation of velocity over the cross-section of the stream, the depth calculated from Equation (1) can be brought still more closely to approximate to the measured depths.

Mr. Bush. EDWARD W. BUSH,† M. AM. SOC. C. E. (by letter).—Figs. 23 to 27 may be of interest to those studying the hydraulic jump below ogee-faced dams, where the toe is some distance up from the bed of the

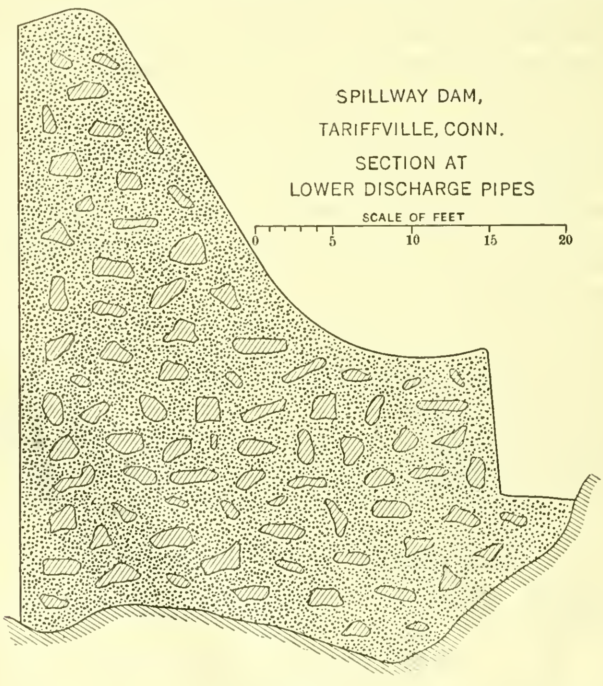

FIG. 23.

stream. This dam is on the Farmington River, at Tariffville, Conn., and was completed late in the season of 1899, by the Hartford Electric Light Company. On February 14th, 1900, when the flood pictures

* *Minutes of Proceedings*, Inst. C. E., Vol. CXCVII, p. 233, *et seq.*

† Hartford, Conn.

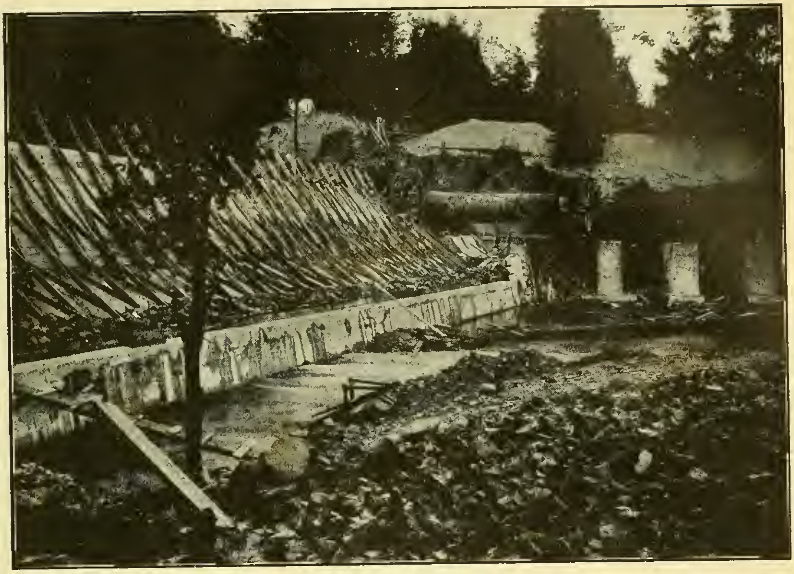

Fig. 24.—Dam at Tariffville, Conn., Under Construction September 4th, 1899.

Fig. 25.—Tariffville Dam, with 9.5-Foot Flood.

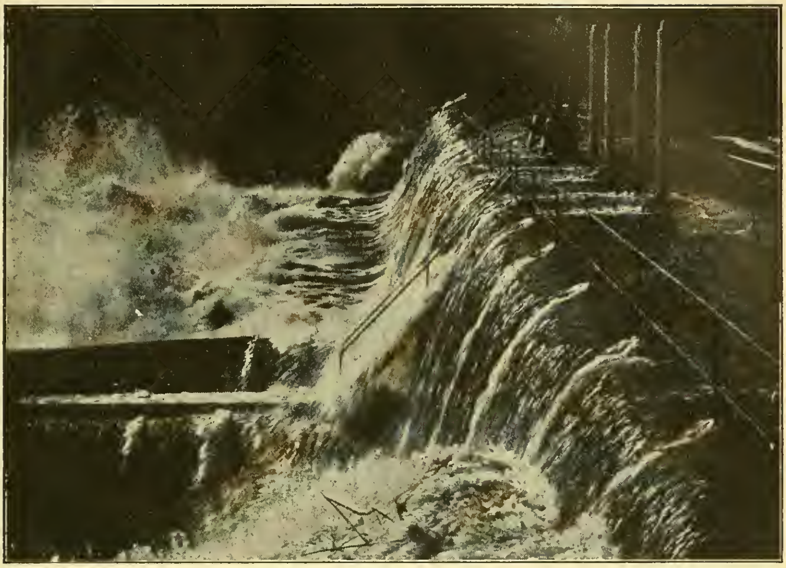

FIG. 26.—TARIFFVILLE DAM, WITH 9.5-FOOT FLOOD.

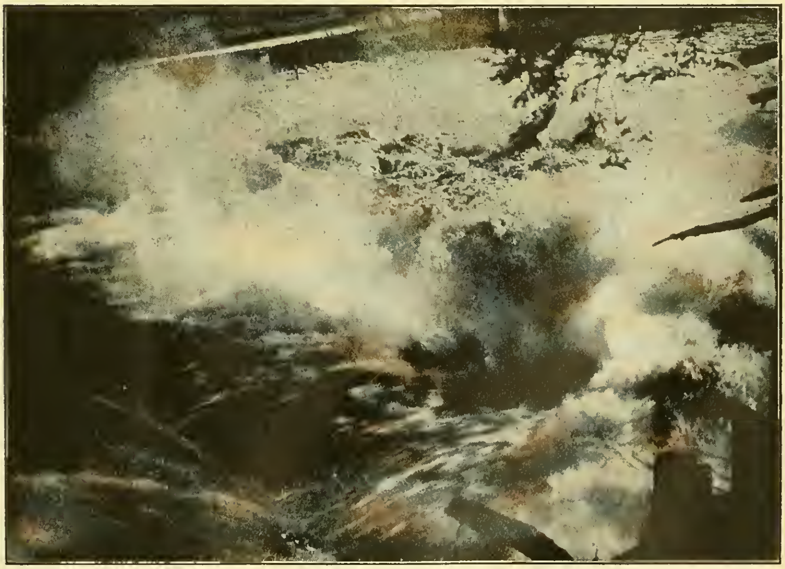

FIG. 27.—TARIFFVILLE DAM, WITH 9.5-FOOT FLOOD.

(Figs. 25, 26, and 27) were taken, the flood height was 9.5 ft. above the crest, and 1.5 ft. above the bulkhead. Mr. Bush.

Fig. 23 shows the section of the dam at the lower discharge pipes; Fig. 24 is a construction view on September 24th, 1899; and Figs. 25, 26, and 27, show the 9.5-ft. flood passing over the dam.

The jump was quite irregular in action, and surged up and down stream over a considerable distance. The height of the jump also varied greatly.

Fig. 24 shows that the river bottom was quite irregular below the dam. This, no doubt, helped to cause the irregularities in the jump. No damage was caused by the flood. The bulkhead wall was raised a few feet to prevent future floods from over-topping it.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

A STUDY OF THE DEPTH OF ANNUAL EVAPORATION FROM LAKE CONCHOS, MEXICO

Discussion.*

By Messrs. T. K. Mathewson, E. F. Chandler, William S. Post, John E. Stirling Thorpe, and Charles H. Lee.

T. K. Mathewson,† M. Am. Soc. C. E. (by letter).—In December, 1911, the writer investigated a proposed hydro-electric power project on the lower reaches of the Conchos River, and was confronted with the almost complete absence of meteorological data in Chihuahua noted by the authors. He also visited the site of the Conchos Dam at that time. This work was in its early stages, and was being carried on by the contractors, S. Pearson and Son. Mr. Mathewson.

In the course of the writer's work for the Guanajuato Power and Electric Company, the depth of yearly evaporation in large reservoirs, decided on and used in a long series of storage investigations in the State of Michoacán, was 1.45 m. (4.75 ft.). This agrees closely with the 1.40 m. (4.59 ft.) determined and used at Lake Conchos. The elevation above sea level of that part of Michoacán is about 2 000 m. (6 560 ft.). The average yearly rainfall from 1892 to 1913, inclusive, was 705.6 mm. (27.78 in.).

Michoacán has the best meteorological records of any of the Mexican States in which the writer has worked. A meteorological observatory was established at Morelia, the State capital, in May, 1908, in charge of Mr. Reyes, a young man of great ability and enterprise. At each county seat there is a well-equipped station, with a volunteer observer to record the rainfall, temperature, and wind. The writer prizes a gift of the complete files of the monthly bulletins issued by the observatory, which he has had bound into a volume. In May, 1912,

* This discussion (of the paper by Edwin Duryea, Jr., M. Am. Soc. C. E., and H. L. Haehl, Assoc. M. Am. Soc. C. E., published in September, 1915, *Proceedings*, and presented at the meeting of November 17th, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Muscatine, Iowa.

Mr. Mathewson. the whole service was stopped by the Revolution, but data of great value were collected and tabulated during the four years of its existence.

Not far from Morelia is a hydro-electric power plant of the Guanajuato Company, where records of rainfall and temperature are kept. Besides being the State capital, Morelia is a cathedral city, and the records of the church meteorological observatory go back to January, 1892. By combining the records of the church, the State, and the Power Company, there is a continuous record of rainfall, by months, in this part of Michoacán for more than 23 years.

The evaporimeter at Morelia is a land-pan. It is not protected from sunshine or wind, but the bottom and sides are protected by the earth and sod. In order to make practical use of the record of this evaporimeter, it was desired to know what proportion of the annual depth of evaporation shown by it might be expected in a large body of water. Very little information could be found on the subject, although several investigators gave values of from one-half to three-fourths.

A farm reservoir was found, having neither inflow nor outflow during a long period of time in the dry season, and a record of the water level was kept, proper correction being made for the only rain storm which occurred. The soil was impervious, so that the fall in the water level could be due to nothing but evaporation. The long record of the evaporimeter was compared with the relatively short record at the reservoir, and a relationship established, which the writer remembers as 56%, instead of the 62% found at Lake Conchos. At any rate, an annual depth of 1.45 m. (4.75 ft.) was determined for the reservoir, as compared with a depth of nearly 3 m. (9.84 ft.) by the evaporimeter. These figures are from memory and a few scanty notes, as the writer was unable to bring his books and papers with him on leaving Mexico, and they are not now accessible.

The thorough and extensive study made by the authors at Lake Conchos is a real contribution to knowledge in a field that has not been investigated very much until recently. The Mexican Northern Power Company may well be congratulated that it has been able to carry its great work to completion during such troublous times.

Mr. Chandler. E. F. Chandler,* Assoc. M. Am. Soc. C. E. (by letter).—The conclusions drawn by the authors from this interesting and comprehensive investigation seem, in general, to be well founded and safely applicable in the region studied. The writer, however, desires to emphasize one point mentioned only briefly, and to call attention to the impropriety, and indeed danger, of applying the numerical results embodied in its conclusions to any distant region of different topography and climate.

Rainfall varies much from year to year, so that the maximum annual rainfall is three times the minimum annual rainfall, more

* University, N. Dak.

Mr. Chandler.

or less, depending on the climatic conditions of the region. The variation in the total annual run-off of any stream may be expected to be very much greater, comparatively, even than that in the annual rainfall of its drainage area. The variations in mean annual temperature and in total annual evaporation at any point, however, are comparatively small, and if an approximately correct relation between the temperature and the evaporation can be discovered, it will be of much use for preliminary estimates where records of evaporation are lacking. Fortunately, long-extended temperature records exist at very many more points and at points more widely distributed than evaporation records.

Since April, 1905, the writer has supervised the continuous maintenance, through the open season of each year, of an evaporation record at University, N. Dak. Daily observations begin soon after the spring break-up of the streams, and continue until they freeze in the fall. There have been only a few very short interruptions, so that the total record covers more than 67 months in the 10 years from 1905 to 1914, inclusive. The latitude of the station is 48°, its longitude 97°, and its altitude 820 ft. above sea level. The evaporation is from a floating-pan, 3 ft. square and 18 in. deep, of the standard U. S. Geological Survey pattern, in a narrow pool or reservoir having an area of about 2 acres and a maximum depth of 10 ft., formed by damming a small stream flowing through a shallow depression in the level Dakota prairie. Conditions are such that the temperature of the water in the pan is not ordinarily found to vary appreciably from that in the surrounding reservoir.

The monthly mean temperatures for each month, from April to November, since records were begun at this station, have varied (for those months when records for the entire month were obtained) from 39 to 73° Fahr., and the evaporation for the month from 1.29 to 7.09 in. The mean evaporation for the period of record for the 10 years, averaging 205 days (from April 14th to November 6th), has been 27.05 in. The mean evaporation for the 6 months (from April 1st to October 31st) has been 25.2 in., and the mean temperature for that period 60° Fahr.

If a table is made showing the mean temperatures and evaporations for the 59 complete months included in the period and for the 12 incomplete months in which at least 10 days' record was secured (these incomplete months being filled out by proportion), and if from this table of 71 values for this station, a diagram (Fig. 16) is prepared according to the method of the authors, it is found to be satisfied fairly well from temperatures 35 to 75° Fahr. by the "straight-line" curve representing the equation,

$$E = 0.135\, T - 4.00,$$

Mr. Chandler.

in which E is the evaporation, in inches, during the month, and T the mean temperature of the month, in degrees, Fahrenheit.

(The curve ought properly to depart somewhat from a straight line for low temperatures, perhaps below 40°, because there is always some evaporation, even at the lowest temperatures, 50 or 60° below the freezing point; but as the evaporation then is small and is measured with difficulty, it has not been determined for these records, and consideration of it here will be omitted.)

It is evident that this well-founded record, maintained according

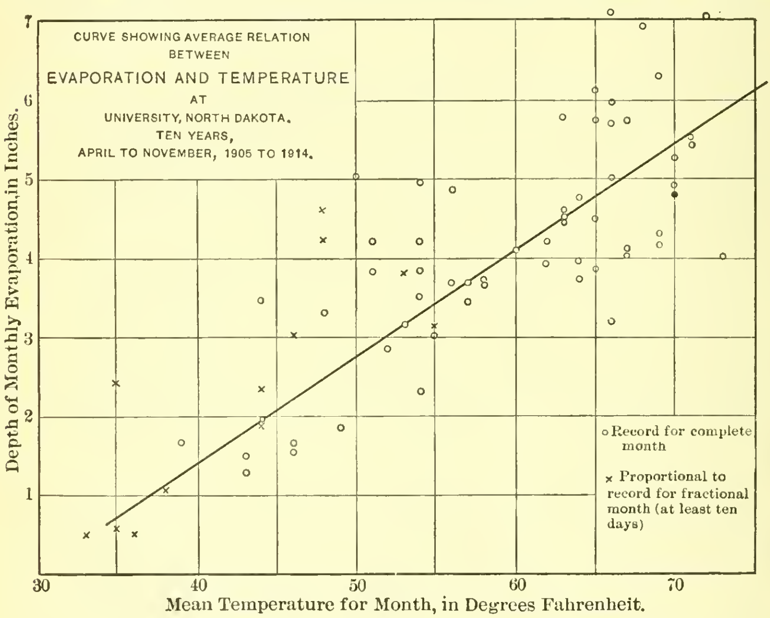

FIG. 16.

to the standard methods, does not in the slightest conform to the numerical conclusions drawn for the Great Plateau in Texas and New Mexico. If the conclusions deduced by the authors for that region were applicable in the Red River Valley of North Dakota, we might use or extend the authors' diagram (a), Fig. 4, and Fig. 13, from which, instead of the equation previously stated, we could deduce, approximately,

$$E = 0.20\ T - 10.00,$$

and then, by computing for each month here, from its mean temperature, the expected evaporation, and adding, we should obtain for the 6 months, from April 1st to October 31st, a total computed evaporation of only about 11 in., instead of the 25 in. that actually occurs.

This disagreement is so great as to corroborate strikingly the statement of the authors immediately after Tables 23 and 24, that apparently the tables are not applicable with safety to places outside of the Great Plateau. Mr. Chandler.

Any attempt to use directly and without change the tables presented in this paper for the purpose of estimating evaporation in other distant portions of the Continent would be likely to lead to incorrect and inexcusably absurd results. If good records of evaporation are available, extending through a period of several years, at some point not remote from the region where estimates are needed, it seems that the methods illustrated so clearly and completely in this paper will provide excellent means for making the corrections needful for the differences in evaporation, caused by comparatively small differences in elevation and in mean temperature; but these methods are not suited for the transfer of figures from actual records to far distant regions, or to points where the mean temperature or elevation is, in great degree, different.

William S. Post,* Assoc. M. Am. Soc. C. E. (by letter).—The writer welcomes the appearance of this paper, particularly as to the conclusions of Table 2. The fact that a floating or a land-pan shows an evaporation in excess of that of a "full-scale" reservoir is confirmed by observations, conducted under the writer's direction, for the projected reservoirs of the Volcan Land and Water Company, in San Diego County, California. Mr. Post.

As the manuscript of an article on "Evaporation in Southern California" had been partly prepared, it has been put in order and is given as a "discussion" in confirmation of the conclusions and method of the authors. The basic data are so scattered through literature that it seems appropriate to contribute and incorporate the following unpublished information from neighboring territory with the observations in the paper.

The results of the writer's study in Southern California, put in the form of Table 2, as tentative conclusions, are as follows:

2 (*b*)—In Southern California the total annual evaporation from a floating-pan is apparently very nearly equal to that from a land-pan, but it shows decided variations, month by month, during the season for the ratio, $\frac{\text{floating-pan}}{\text{land-pan}}$, somewhat as follows:

December, January, and February	about	150	per	cent.
March, April, May, and June	"	108	"	"
July, August, September, October, and November	"	90	"	"
Mean (33 months compared, of which 5 are for a pair of pans close together, and				

* San Diego, Cal.

Mr. Post.

the rest for pans 2 miles apart, and not under precisely identical conditions)....about 99 per cent.

Mean of foregoing pair of pans close together, 5 months (May to September, inclusive).. " 98 " "

2 (c)—Evaporation depth from a large reservoir equals about 81% of that from a 3-ft. floating-pan.

CURVES SHOWING AVERAGE RELATIONS BETWEEN EVAPORATION AND TEMPERATURE IN SAN DIEGO COUNTY, CALIFORNIA.
OBSERVATIONS BY VOLCAN LAND AND WATER COMPANY.

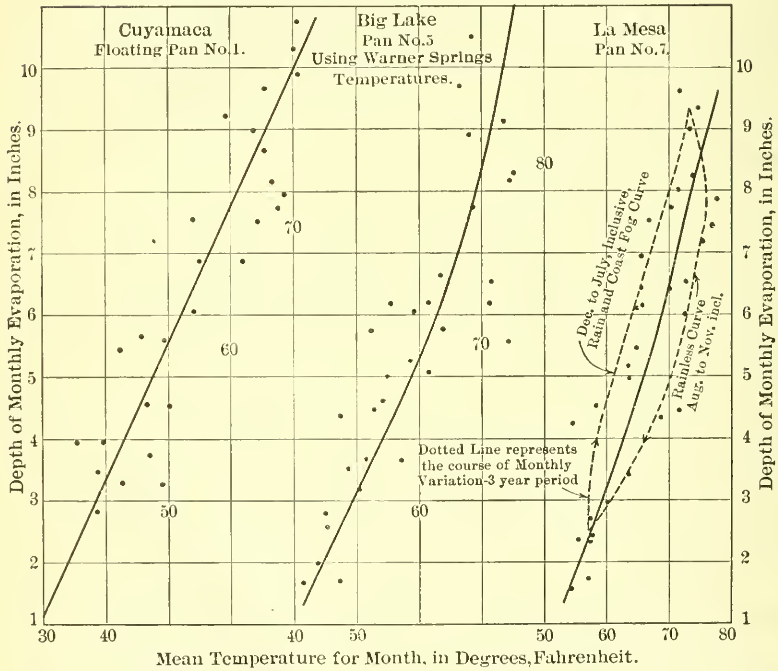

FIG. 17.

DETAIL OF STUDY OF EVAPORATION IN SOUTHERN CALIFORNIA.

The Volcan Land and Water Company and the Cuyamaca Water Company have maintained eleven pans in various locations in San Diego County, which were placed and observed under the writer's direction. These pans are all a standard Weather Bureau type, 3 ft. square and 1½ ft. deep, except certain temporary ones which are 9 in. square. Rain gauges are placed near each pan.

Description of Pans.

Mr. Post.

Pan No. 1.—West margin of Cuyamaca Lake, near the divide between the Pacific Ocean and the Salton Basin, about 40 miles from the Coast. Altitude, 4 620 ft. In the winter the prevailing winds, rain-bearing, are from the southwest, clearing from the north; in summer, desert conditions exist, varying from summer thunder storms of the Arizona type in July and August to extremely dry southeast winds in September and October.

The pan floats in the water, protected by a boom and in addition by a V-shaped shield to prevent wave action. This shield undoubtedly affects the standardization of the results, probably increasing the evaporation.

Pan No. 2.—Three miles east of Pan No. 1, on easterly shore line of Cuyamaca Lake, 10 ft. above high-water line, flush with the natural soil. It is normally 20 ft. above the lake level. It is also nearer the desert influences. Altitude, 4 640 ft.

Pan No. 4.—"In water", floating in the San Luis Rey Creek. Altitude, 2 620 ft.

Pan No. 5.—Four miles west of Warner Hot Springs Post Office. It is "in water", floating on the surface of Big Lake, some 50 acres in extent. Altitude, 2 780 ft.

Pan No. 6.—Was near Pan No. 5, and set in moist spring-fed ground. It was moved in May, 1915, to ordinary ground, in order to secure uniformity and comparison with other pans "in ground", and thereafter called Pan No. 6-*A*. Altitude, 2 800 ft.

Pan No. 7.—Floating in La Mesa Reservoir, 5 miles east of San Diego. Altitude, 480 ft.

Pan No. 8.—In the ground, 4 miles north of Ramona, at Pamo Reservoir Site. Altitude, 900 ft.

Pan No. 9.—In the ground, 10 miles northeast of Ramona, at Sutherland Reservoir Site. Altitude, 2 100 ft.

Pan No. 10.—In the ground, 4 miles west of Ramona, at Santa Maria Reservoir Site. Altitude, 1 400 ft.

Pan No. 11.—In the ground, 5 miles southwest of Escondido, at Carroll Reservoir Site. Altitude, 230 ft.

Pans Nos. 8, 9, and 10 were 9 in. square until December, 1914, when standard 3-ft. square pans were put in.

Data have also been published in *Bulletin No. 81,* United States Geological Survey, Water Supply Papers, for the Sweetwater Reservoir; and the elaborate observations of C. H. Lee, Assoc. M. Am. Soc. C. E., in *Bulletin No. 294;* U. S. Geol. Survey, Water Supply Papers, showing the determinations of the Los Angeles Aqueduct near Independence, Cal.

Mr. Post.

TABLE 51.—GROSS EVAPORATION RECORDS. FLOATING 3-FT. SQUARE PANS. VOLCAN LAND AND WATER COMPANY.

ELEVATION.	4 620 FT.		2 620 FT.		2 780 FT.		480 FT.	
	Pan No. 1.		Pan No. 4.		Pan. No. 5.		Pan No. 7.	
Month.	Evaporation, in inches.	Mean temperature.	Evaporation, in inches.	Mean temperature.	Evaporation, in inches.	Mean temperature.*	Evaporation, in inches.	Mean temperature.
January, 1913	....	34.8	(3.00)	39.8	(2.00)	42.9	1.59	54.1
February, 1913	....	37.5	4.12	44.8	3.30	44.4	4.23	54.5
March, 1913	....	39.2	3.79	47.8	4.31	47.5	4.51	58.4
April, 1913	....	48.6	4.17	50.8	5.71	52.6	6.19	64.3
May, 1913	....	56.1	3.89	55.7	6.03	59.3	7.54	66.9
June, 1913	9.25	59.0	4.41	60.0	5.76	63.9	6.98	65.7
July, 1913	8.18	66.6	4.42	70.2	6.51	71.6	9.64	71.7
August, 1913	7.94	68.2	3.72	72.0	5.53	74.2	8.45	76.8
September, 1913	9.68	65.4	4.01	68.2	6.18	71.3	8.21	75.4
October, 1913	6.86	54.8	4.93	59 ±	6.29	61.7	6.46	70.2
November, 1913	5.63	45.1	3.30	49.5	3.67	51.7	3.42	63.8
December, 1913	2.85	38.5	3.40	42.2	2.79	45.0	2.70	57.3
	....	51.2	47.16	55.0	58.08	57.2	69.92	64.9
January, 1914	3.28	42.4	2.76	46.0	1.69	47.7	2.37	57.6
February, 1914	5.46	42.0	(3.20)	47.5	3.50	49.0	2.94	60.1
March, 1914	7.20	47.4	4.11	51.2	4.60	54.2	6.15	65.2
April, 1914	5.60	49.4	4.24	54.1	4.98	55.2	6.16	65.7
May, 1914	7.53	53.9	4.87	57.0	5.23	58.9	5.48	64.9
June, 1914	8.99	63.4	6.31	64.3	6.61	63.7	7.76	70.4
July, 1914	9.90	70.6	6.70	71.3	8.17	74.5	9.03	73.2
August, 1914	10.76	70.3	8.38	70.4	9.10	73.3	8.26	73.5
September, 1914	7.53	64.0	5.86	65.6	7.75	68.5	6.54	72.7
October, 1914	6.08	54.0	3.46	57.7	5.04	61.7	4.47	71.5
November, 1914	3.28	48.9	2.95	53.3	3.62	57.3	4.34	68.7
December, 1914	3.94	35.2	1.38	41.2	1.65	41.6	2.57	55.6
	79.55	53.5	54.22	56.6	61.94	58.8	65.87	66.6
January, 1915	3.47	38.5	1.75	42.4	1.96	43.8	1.73	57.0
February, 1915	3.97	39.6	(3.00)	44.6	2.51	45.1	2.40	57.7
March, 1915	4.56	46.3	4.62	49.2	3.16	50.6	4.99	63.7
April, 1915	3.75	47.8	4.85	53.3	4.44	52.9	5.15	63.5
May, 1915	4.52	50.0	4.45	55.4	6.17	55.7	6.45	65.5
June, 1915	9.66	65.0	9.21	63.5	9.68	66.6	8.01	71.6
July, 1915	8.74	68.5	10.45	68.0	10.46	68.4	9.34	74.8
August, 1915	10.30	69.8	8.70	71.5	8.28	75.2	7.89	77.5
September, 1915	6.88	61.7	7.11	63.0	8.90	68.0	6.02	72.7

NOTE.—Figures in parentheses are interpolated from other pans.

* Mean temperature from Warners Spring, 4 miles east.

Mr. Post.

TABLE 52.—GROSS EVAPORATION RECORDS. 3-FT. LAND-PANS, EXCEPT AS NOTED. VOLCAN LAND AND WATER COMPANY.

ELEVATION.	4 640 FT.		2 800 FT.		2 800 FT.		900 FT.	2 100 FT.	1 400 FT.	230 FT.	
	Pan No. 2.		Pan No. 6 ‖ (*E*)		Pan No. 6-*A*.		Pan No. 8.	Pan No. 9.	Pan No. 10.	Pan No. 11.	
Month.	Evaporation, in inches.	Mean temperature.*	Evaporation, in inches.	Mean temperature.†	Evaporation, in inches.	Mean temperature.†	Evaporation, in inches.	Evaporation, in inches. §	Evaporation, in inches.	Evaporation, in inches.	Mean temperature.‡
1913											
January...	....	34.8	(2.00)	42.9	......	......	....	......	....	....	....
February..	....	37.5	(3.00)	44.4	......	......	....	......	....	....	....
March......	....	39.2	3.58	47.5	......	......	....	......	....	....	....
April	4.87	48.6	4.97	52.6	......	......	....	......	....	....	....
May........	8.38	56.1	4.96	59.3	......	......	....	......	....	....	....
June........	8.90	59.0	4.66	63.9	......	......	....	......	....	....	....
July........	8.99	66.6	6.67	71.6	......	......	....	......	....	....	....
August....	8.96	68.2	5.85	74.2	......	......	....	......	....	....	....
September	9.39	65.4	6.73	71.3	......	......	....	......	....	....	....
October...	7.49	54.8	4.90	61.7	......	......	....	......	....	....	....
November.	4.40	45.1	2.82	51.7	......	......	....	......	....	....	....
December.	2.30	38.5	1.93	45.0	......	......	....	......	....	....	....
	63.68	51.2	52.07	57.2	......	......	....	......	....	....	....
1914											
January...	2.30	42.4	1.70	47.7	......	......	2.79	1.58	3.03	....	....
February..	3.32	42.0	2.67	49.0	......	......	2.56	2.43	2.63	....	....
March.....	5.26	47.4	3.49	54.2	......	......	3.51	3.33	4.03	....	....
April	5.50	49.4	3.07	55.2	......	......	3.56	3.58	4.88	....	....
May........	6.19	53.9	3.30	58.9	......	......	4.72	3.73	4.58	....	....
June.......	10.04	63.4	4.34	63.7	......	......	8.23	6.06	8.83	....	....
July........	12.23	70.6	5.05	74.5	......	......	10.26	7.65	8.77	....	....
August....	12.42	70.3	4.97	73.3	......	......	9.92	7.83	8.23	....	....
September.	8.93	64.0	4.53	68.5	......	......	6.68	5.78	6.77	....	....
October....	6.76	54.0	4.36	61.7	......	......	4.51	6.52	5.49	....	....
November.	4.20	48.9	3.42	57.3	......	......	3.16	4.42	4.69	....	....
December..	1.72	35.2	1.87	41.6	......	......	1.40	3.07	1.85	....	....
	78.87	53.5	42.77	58.8	......	......	61.30	55.98	63.78	....	....
1915											
January ...	2.93	38.5	1.63	43.8	......	......	1.44	1.44	1.61	1.07	48.9
February..	3.12	39.6	2.70	45.1	......	......	2.00	......	2.42	1.15	51.2
March......	4.77	46.3	3.02	50.6	......	......	3.07	3.01	3.33	3.06	57.5
April.......	4.29	47.8	4.25	52.9	......	......	5.29	3.09	3.00	2.45	59.8
May........	4.92	50.0	....	......	4.73	55.7	4.00	3.65	3.75	3.92	62.1
June.......	7.94	65.0	....	......	7.86	66.6	6.18	5.36	5.83	5.89	69.1
July.......	10.22	68.5	....	......	12.01	68.4	6.52	6.02	6.62	6.83	72.8
August....	11.14	69.8	....	......	9.89	75.2	6.62	6.08	6.93	6.77	73.9
September.	7.80	61.7	....	......	9.41	68.0	5.01	4.50	5.00	4.90	....

* Mean temperature at Pan No. 1.

† Mean temperature from Warners Springs, 4 miles east.

‡ Mean temperature from Escondido, 5 miles northeast.

§ 9-in. square pan, January to November, inclusive, 1914. 3-ft. square, standard pan, December, 1914 and 1915.

‖ Pan No. 6 in moist spring-fed ground discontinued.

Mr. Post. The annual totals are incorporated in the summary, Table 53, under the captions "Sweetwater Reservoir" and "Independence".

"FULL-SCALE" DATA OF ACTUAL GROSS RESERVOIR EVAPORATION.

In the application of such data to hydrographic studies, it is clear that the ratio which a pan bears to a "full-scale" reservoir should be determined. This discussion may be considered as a progress report on this ratio.

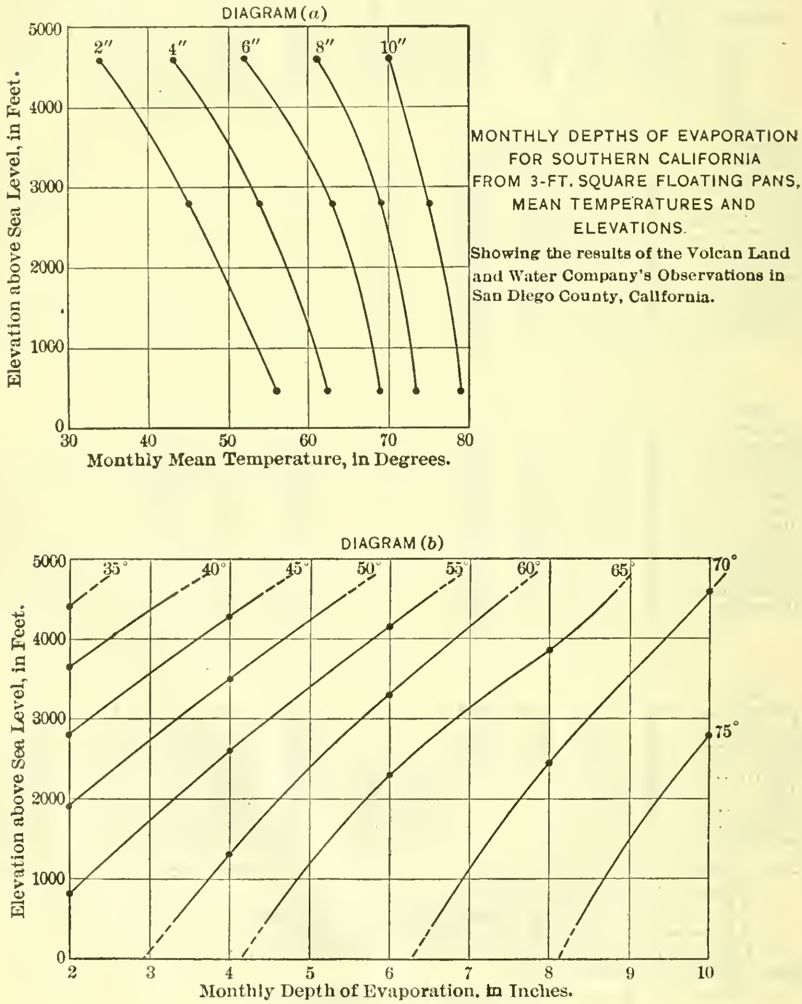

FIG. 18.

TABLE 53.

Mr. Post.

Year.	Independence. Altitude = 3 760 ft.	Sweetwater. Altitude = 200 ft.
1889	......	57.35
1890	......	59.85
1891	......	58.08
1892	......	59.55
1909	69.05	......
1910	65.44	......
1912	......	......
1913	......	......
1913	......	......
1914	......	......
Averages	67.25	58.70

TABLE 54.—Determination of Actual Net Evaporation on Cuyamaca Lake (Including the Effect of "Make-up" or Drainage from Moist Border Land as Reservoir Lowers).

Note.—During the months of high rainfall—December to April—it is impossible to make this comparison because of the unknown and large run-off. With the cessation of rains, the surface inflow can be closely observed, and is of a quantity sufficiently small, compared with the large area of the reservoir, to be almost negligible.

Period.	Average number of acres exposed.	Increase + or decrease − of reservoir, in acre-feet.	Accounted For: Draft, in acre-feet.	Net depletion, in acre-feet.	Run-off into lake, in acre-feet.	Direct rainfall, in acre-feet.	Gross evaporation, in acre-feet.	Monthly gross evaporation, in inches.
1912.								
June 1-30	472	−542	195	−347	23	12	382	9.68
June 30-July 27	414	−627	541	− 86	11	14	111	3.57
July 27-August 31	332	−747	523	−224	14	17	255	8.39
August 31-October 1	250	−419	290	−129	6	0	135	6.48
May, 1913	418	−263	47	−216	24	12	242	7.27
June, 1913	361	−518	385	−133	18	12	163	5.24
July, 1913	283	−600	531	− 69	19	15	93	4.05
August, 1913	220	−353	381	+ 28	45	33	50	2.78
September, 1913	145	−369	329	− 40	12	1	53	4.39
March, 1914	380	− 8	0	− 8	84	54	146	4.63
April, 1914	384	+ 67	0	+ 67	68	63	64	2.00
May, 1914	380	−199	0	−199	20	5	224	7.08
June, 1914	356	−255	0	−255	12	11	278	9.30
July, 1914	285	−452	280	−187	14	22	223	8.85
August, 1914	228	−472	275	−197	12	0	209	11.00
September, 1914	155	−282	215	− 67	6	4	77	5.98
June, 1915	844	−563	0	−563	12	0	575	8.16
July, 1915	816	−829	131	−698	12	0	816	10.30
August, 1915	777	−860	422	−438	19	75	532	8.28
September, 1915	733	−836	391	−445	12	0	457	7.48

Mr. Post.

TABLE 55.—CORRECTION OF SWEETWATER EVAPORATION DATA TO REMOVE 5% SYSTEM LOSS INCLUDED IN EVAPORATION AS GIVEN BY MR. JOHN F. COVERT.

Quantities, in feet.

Month.	1910.			1911.			1912.			1913.			1914.		
	As given by Covert.	Losses 5%.	True evaporation.	As given by Covert.	Losses 5%.	True evaporation.	As given by Covert.	Losses 5%.	True evaporation.	As given by Covert.	Losses 5%.	True evaporation.	As given by Covert.	Losses 5%.	True evaporation.
January	0.200	0.001	0.199	0.260	0.006	0.257	0.130	0.002	0.128	0.356	0.015	0.341	0.148	0.008	0.140
February	0.200	0.003	0.197	0.200	0.001	0.199	0.330	0.021	0.309	0.195	0.006	0.189	0.204*	....	0.174
March	0.270	0.009	0.261	0.270	0.002	0.268	0.170	0.016	0.154	0.266*	....	0.226	0.266*	....	0.227
April	0.320	0.009	0.311	0.350	0.032	0.318	0.480	0.004	0.476	0.347*	....	0.295	0.347*	....	0.295
May	0.650	0.111	0.539	0.610	0.047	0.563	0.410	0.015	0.395	0.654	0.076	0.578	0.531	0.063	0.468
June	0.590	0.036	0.554	0.630	0.046	0.574	0.740	0.059	0.681	0.344	0.065	0.279	0.811	0.093	0.718
July	0.720	0.038	0.682	0.670	0.049	0.621	0.680	0.056	0.624	1.053	0 076	0.977	0.785	0.111	0.674
August	0.610	0.027	0.583	0.660	0.055	0.605	0.760	0.068	0.692	0.718	0.092	0.626	0.734	0.134	0.600
September	0.440	0.028	0.412	0.690	0.045	0.645	0.600	0.060	0.540	0.674	0.096	0.578	0.328	0.158	0.170
October	0.510	0.017	0.493	0.500	0.044	0.456	0.460	0.049	0.411	0.459	0.078	0.381	0.368	0.100	0.268
November	0.360	0.013	0.347	0.380	0.032	0.348	0.370	0.046	0.324	0.310	0.042	0.268	0.084	0.072	0.012
December	0.380	0.000	0.380	0.340	0.018	0.322	0.080	0.031	0.049	0.106	0.018	0.088	0.089	0.021	0.068
Totals	5.25	0.292	4.958	5.55	....	5.176	5.21	....	4.783	5.482	....	4.826	....	4.695	3.814

* Pan records used: revised by taking 85% of pan records.

Cuyamaca Reservoir.—It was hoped that a comparison of the reservoir stages at Cuyamaca Lake, accounting for inflow and draft, would establish this ratio. A decided practical difficulty has been met in considering the "make-up" of the reservoir, which is very flat, and a slight change in level undoubtedly drains a large area of moist lands. However, it serves as an example of the actual net evaporation, including "make-up". This is somewhat compensated by seepage from the dam. Mr. Post.

In so far as rising or stationary conditions are incorporated in the calculation, "make-up" is neutralized. It is neglected as a factor when averaging the 13 months of observations.

Sweetwater Reservoir.—Mr. John F. Covert, Engineer of the San Diego Land and Town Company, has computed the gross evaporation of Sweetwater Reservoir, making all allowances except for losses in the distribution system, as he was obliged to use the summation of the meters of the distribution system to obtain the draft.

This distribution loss is assumed by the writer to be 5%, and Mr. Covert's data have been revised to this extent.

TABLE 56.—Sweetwater Gross Evaporation, in Inches (Finally Corrected).

Month.	1910.	1911.	1912.	1913.	1914.	Mean.	Percentage.
January	2.39	3.09	1.54	4.10	1.68	2.56	4
February	2.36	2.39	3.70	2.26	2.09	2.56	5
March	3.15	3.21	1.85	2.71	2.73	2.73	5
April	3.74	3.80	5.71	3.54	3.54	4.08	7
May	6.43	6.78	4.75	6.91	5.60	6.09	11
June	6.68	6.90	8.19	3.35	8.60	6.74	12
July	8.20	7.48	7.50	11.70	8.10	8.80	15
August	7.00	7.26	8.31	7.50	7.20	7.45	13
September	4.94	7.73	6.49	6.91	2.04	5.62	10
October	5.91	6.65	4.94	4.58	3.21	5.06	9
November	4.16	4.17	3.89	3.21	0.14	3.11	5
December	4.56	3.86	0.59	1.06	0.83	2.18	4
Totals	59.52	63.32	57.46	57.83	45.76	56.98	100

Other "Actual" Comparisons.—On Murray Hill Reservoir, near Grossmont, valves were closed, and there was no inflow during the period from April 26th 3.30 P. M. to 7.30 A. M. May 2d, 1913.

Drop in lake....................1.60 in.

The rate is.......................$\frac{1.60}{5.68}$ = 0.282 in. per day.

Mr. Post. At La Mesa, floating-pan No. 7, 5 miles distant, during the same period, the decrease of water in the pan from April 25th (noon) to May 2d (noon) .... 1.76 in.

Subtract $1\frac{1}{3}$ days excess of period at rate of 0.200 per day, as per preceding week at La Mesa .... 0.27 "

1.49 in.

$$\frac{\text{Actual}}{\text{Pan}} = \frac{1.60}{1.49} = 107 \text{ per cent.}$$

In this instance the lake shows a greater evaporation than the pan.

Upper Otay Reservoir.—This is a small water-shed with practically no run-off during the months given, and no draft, except some domestic use on the Babcock place, which is not known, but probably not great.

TABLE 57.—OBSERVATIONS BY CITY OF SAN DIEGO CARETAKERS.

(Probably rather roughly to the nearest inch.)

Month.	Fall in reservoir, in inches.	Rainfall,† in inches.	Evaporation, in inches.
June, 1914	6.64*	0	6.64
July, "	8 75	0	8.75
Aug., "	12.00	0.06†	12.06
June, 1915	6.00	0	6.00
July, "	8.00	0	8.00
Aug., "	10.00	0	10.00
Sept., "	5.00	0	5.00

* Interpolated 1 day.

† At Sweetwater Reservoir, 5 miles distant.

TABLE 58.—DETERMINATION OF THE RATIO BETWEEN ACTUAL EVAPORATION AND FLOATING-PAN.

Month.	Cuyamaca Reservoir. Actual as computed from Table 54.	Floating-pan No. 1.	Ratio.
June, 1913	5.24	9.25	57%
July, 1913	4.05	8.18	49%
August, 1913	2.78*	7.94*	35%
September, 1913	4.39	9.68	45%
March, 1914	4.63	7.20	64%
April, 1914	2.00*	5.60*	36%
May, 1914	7.08	7.53	94%
June, 1914	9 30	8.99	103%
July, 1914	8.85	9 90	89%
August, 1914	11.00	10.76	102%
September, 1914	5.98	7.53	79%
June, 1915	8.16	9.66	84%
July, 1915	10.30	8.74	118%
August, 1915	8.28	10.30	80%
September, 1915	7.48	6.88	108%
	94.74*	114.20*	83%

* August, 1913, and April, 1914, excluded from totals.

Mr. Post.

TABLE 59.—DETERMINATION OF THE RATIO BETWEEN ACTUAL EVAPORATION AND FLOATING-PAN.

Sweetwater Reservoir (Table 56) and Upper Otay Reservoir (Table 57), compared with Pan No. 7 floating on La Mesa Reservoir. These three localities are roughly the same distance back from the Pacific Coast line, respectively, 12, 9, and 12 miles; the altitudes are 200, 500, and 480 ft. Pan No. 7 is the most northerly, Sweetwater Reservoir is distant 7 miles, and Upper Otay Reservoir 5 miles farther.

Month.	Actual.			Pan No. 7.
	Sweetwater.	Upper Otay.	Mean.	
January, 1913	4.10	....	4.10	1.59
February, 1913	2.26	....	2.26	4.23
March, 1913	2.71	....	2.71	4.51
April, 1913	3.54	....	3.54	6.19
May, 1913	6.91	....	6.91	7.54
June, 1913	3.35	....	3.35	6.98
July, 1913	11.70	....	11.70	9.64
August, 1913	7.50	....	7.50	8.45
September, 1913	6.91	....	6.91	8.21
October, 1913	4.58	....	4.58	6.46
November, 1913	3.21	....	3.21	3.42
December, 1913	1.06	....	1.06	2.70
January, 1914	1.68	....	1.68	2.37
February, 1914	2.09	....	2.09	2.94
March, 1914	2.73	....	2.73	6.15
April, 1914	3.54	....	3.54	6.16
May, 1914	5.60	....	5.60	5.48
June, 1914	8.60	6.64	7.62	7.76
July, 1914	8.10	8.75	8.48	9.03
August, 1914	7.20	12.06	9.63	8.26
September, 1914	2.04	....	2.04	6.54
October, 1914	3.21	....	3.21	4.47
November, 1914	0.14	....	0.14	4.34
December, 1914	0.83	....	0.83	2.37
June, 1915	....	6.00	6.00	8.01
July, 1915	....	8.00	8.00	9.34
August, 1915	....	10.00	10.00	7.89
September, 1915	....	5.00	5.00	6.02
	....	....	134.42	167.05 Ratio = 80 per cent.

Mr. Post.

TABLE 60.—DETERMINATION OF THE RATIO BETWEEN FLOATING-PAN AND LAND-PAN.

Pans were 2 miles apart; elevation = 4 600 ft.

Month.	Pan No. 1. Floating.	Pan No. 2. Land.	Ratio.
June, 1913	9.25	8.90	104%
July, 1913	8.18	8.99	91%
August, 1913	7.94	8.96	89%
September, 1913	9.63	9.39	102%
October, 1913	6.86	7.49	92%
November, 1913	5.63	4.40	128%
December, 1913	2.85	2.30	126%
January, 1914	3.28	2.30	138%
February, 1914	5.46	3.82	146%
March, 1914	7.20	5.26	136%
April, 1914	5.60	5.50	102%
May, 1914	7.53	6.19	122%
June, 1914	8.99	10.04	90%
July, 1914	9.90	12.23	89%
August, 1914	10.76	12.42	87%
September, 1914	7.53	8.93	84%
October, 1914	6.08	6.76	90%
November, 1914	3.28	4.20	78%
December, 1914	3.94	1.72	230%
January, 1915	3.47	2.93	118%
February, 1915	3.97	3.12	127%
March, 1915	4.56	4.77	97%
April, 1915	3.75	4.29	87%
May, 1915	4.52	4.92	92%
June, 1915	9.66	7.94	121%
July, 1915	8.74	10.22	85%
August, 1915	10.30	11.14	91%
September, 1915	6.88	7.80	88%
	185.79	186.43	99%

TABLE 61.—DETERMINATION OF RATIO BETWEEN FLOATING-PAN AND LAND-PANS.

Pans adjacent; elevation, 2 800 ft.

Month.	Pan No. 5.	Pan No. 6-*A*.	Ratio.
May, 1915	6.17	4.73	130%
June, 1915	9.68	7.86	115%
July, 1915	10.46	12.01	88%
August, 1915	8.28	9.89	83%
September, 1915	8.90	9.41	94%
	43.49	43.90	99%

TABLE 62.—DISTRIBUTION OF EVAPORATION IN SOUTHERN CALIFORNIA. Mr. Post.

Month.	Sweetwater. (Table 56.)	Sweetwater. (Pans.)	Pan No. 7.	Pan No. 5.	Pan No. 2.
January	4	4	3	4	4
February	5	4	5	5	4
March	5	5	8	7	7
April	7	7	9	8	6
May	11	11	10	10	9
June	12	11	10	10	12
July	15	15	13	12	14
August	13	14	11	12	14
September	10	11	10	12	12
October	9	9	9	10	9
November	5	5	6	6	6
December	4	4	6	4	3
	100%	100%	100%	100%	100%

JOHN E. STIRLING THORPE,* ESQ. (by letter).—Soon after Mr. Duryea was retained as Consulting Engineer of the Mexican Northern Power Company, he asked the writer whether any study had been made of the evaporation of Lake Conchos and, if not, it was his wish that one should be commenced immediately. Subsequently, he outlined in detail the sizes, shape, etc., of the pans he wished to have made, and stated where he wanted the land and floating pans placed. Mr. Thorpe.

Due to the importance of obtaining accurate measurements, great care was taken in making the evaporation pans, especially the floating pans. The high winds and the wide expanse of the lake caused it to become very rough at times, necessitating a very strong pan. The requisite strength was obtained by making a raft of 6 by 8-in. scantling and placing splash-boards around the sides to prevent, as much as possible, the water of the lake from entering the pan. This type is illustrated by Fig. 19. One side of the splash-boards opened like a sliding door. When the observer wished to take measurements, he opened this door, and was able to run the bow of his boat to within a few inches of the pan, and take his measurements without disturbing its equilibrium. In rough weather, the pan was towed into a near-by cove in order that more accurate measurements could be made. The floating pans were anchored, one in the middle of each of the wide areas of the lake.

The first land pans were placed on a base of 2 by 4-in. scantling, with a strip of heavy lagging nailed around the outside of the pan to prevent the rays of the sun from striking the water. Later, this arrangement was changed by Mr. Duryea, the pans being placed directly on the ground, with earth packed around the sides level with their tops. On account of the large numbers of stray animals in this

* Birmingham, Ala.

Mr. Thorpe. section of Mexico, special care was taken to prevent them from drinking any of the water in the pans by constructing a strong fence of 6 by 6-in. scantling and barbed wire. Fig. 20 illustrates one of the land pans, as first placed. This pan was set at Elevation 1 296. Another and similar pan was set at Elevation 1 317.

One of the junior engineers was detailed to take all measurements, keep the pans filled to a depth of approximately 10 in., check up leaks, etc. A watchman also visited the pans several times during the 24 hours to prevent any tampering with them.

A water line was painted on the inside of each corner of the pan, and all four corners were checked up on each measurement of the depth; but the corner marked No. 1 was taken as the master corner, from which all depths were taken.

No trouble was experienced with the land pans, with which great care was taken at all times, and accurate results were obtained. This statement applies also to the floating pans during calm weather; but during rough weather the results from these pans were doubtful and were so treated, careful attention being paid to weather conditions. Four engineers, including the writer, took independent observations each day, and the majority opinion of these observations was briefly recorded each day with the temperature observations taken with maximum and minimum thermometers. At each land pan there was a rain gauge, and this was observed jointly with the pan.

Mr. Lee. CHARLES H. LEE,* ASSOC. M. AM. SOC. C. E. (by letter).—The authors have attempted, through assumptions and highly theoretical methods, carried out in great detail, to estimate annual evaporation from a lake surface more than 300 miles from the nearest point of actual observation. They have also attempted an exhaustive investigation and comprehensive analyses of the problem. They have based much of their framework, however, on a fundamental assumption which the writer believes to be in error. As a result thereof, the computed value of yearly gross evaporation from Lake Conchos, as set forth in Table 4, is far from the truth. The paper will be discussed by considering first each of the subsidiary conclusions and finally the principal conclusion.

During the past 10 years, the writer has had occasion to give considerable study to the subject of evaporation. During the 3 years, 1908 to 1911, he carried on extensive quantitative observations of evaporation from both land and soil pans in Owens Valley, California, and subsequently has made a computation of annual evaporation from Owens Lake, based on detailed observations of inflow, lake level, and lake area, covering a period of 8 years. He has kept in touch with the experiments conducted for the United States Weather Bureau at

* Los Angeles, Cal.

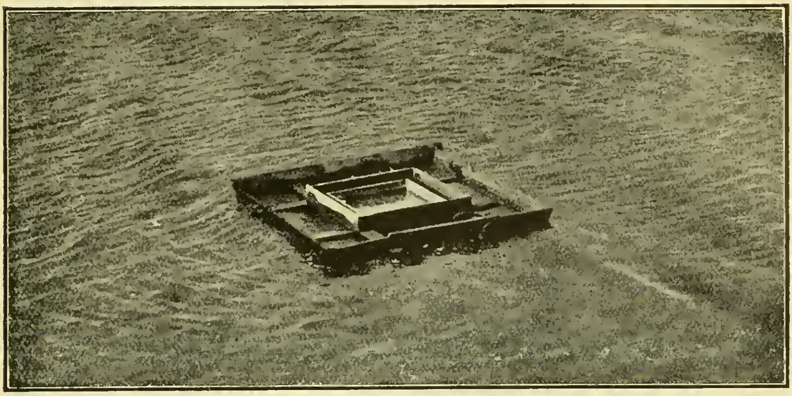

FIG. 19.—TYPE OF FLOATING PAN USED AT LAKE CONCHOS.

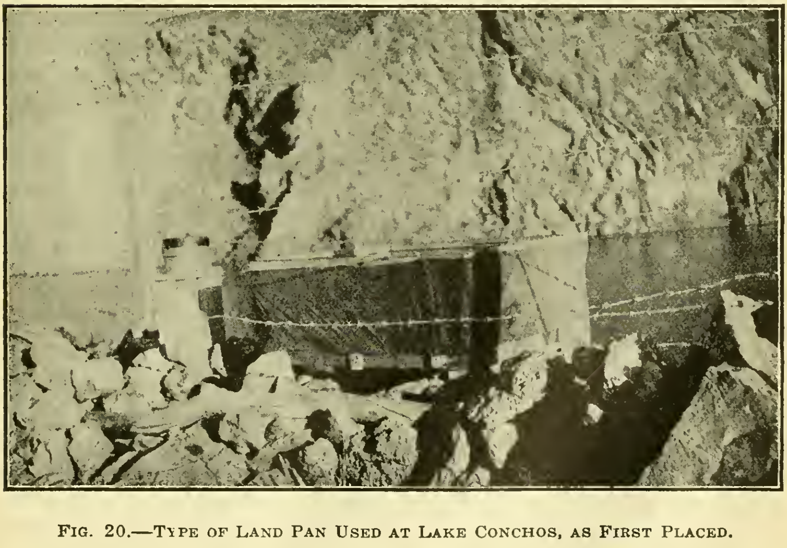

FIG. 20.—TYPE OF LAND PAN USED AT LAKE CONCHOS, AS FIRST PLACED.

Reno and Salton Sea, by Professor Frank H. Bigelow, and has also made a study of the various available records in Western States, both published and unpublished. Mr. Lee.

The subsidiary conclusions will be taken up in order as follows.

(a)—*Relative Evaporation Depths from Pans of Different Sizes.*— The writer wishes to point out, in connection with Professor Bigelow's formula,

$$E_0 = C_2 \times \frac{e_s}{e_d} \times \frac{d_e}{d_s} \times (1 + 0.070\,w),$$

that it represents the evaporation depths from isolated areas of exposed water surface distantly removed from other similar areas. It not only represents evaporation depth from pans, but also from open tanks, pools, ponds, lakes, or seas. The coefficient, C_2, is a variable, depending on the size of the evaporating surface. Small pans, representing one extreme, give large values of C_2. Larger pans give rapidly decreasing values of C_2. Pools and ponds would give less rapidly decreasing values, and large lakes give practically the smallest value, or approximately 0.024. In brief, the equation does not give evaporation depths from pans, only, as one might infer from the authors' statements and from Fig. 5, but from the surfaces of isolated bodies of water of various sizes.

Carrying the idea a little further, the writer would emphasize the fact that the equation does not apply to a pan floating on a lake surface, except as it applies to that surface as a whole. Water isolated in a pan floating on a lake surface is losing by evaporation under the same vapor blanket conditions as the surrounding water surface outside the pan. There may be a difference of temperature between the water inside and outside of such a pan, because of the absorption by the metal of heat from the sun's rays. The writer has observed such temperature differences to be very small, however, and Professor Bigelow states that there is "a small correction due to this difference of temperature".*

The coefficient, C_2, depends on the ease with which saturated vapor from a water surface escapes into the atmosphere. In the case of a small pan not within the vapor blanket of a lake, nothing opposes the dissipation of the vapor in four horizontal directions as well as vertically. In the case of a pan floating on a lake, the lateral expansion of the vapor from the pan is opposed in all directions by the vapor arising from the surrounding water, and the only direction of free dissipation is vertically. A lake, considered as a whole, has not as great an opportunity for lateral dissipation of its vapor as a pan, for the reason that vapor blankets do not increase in depth in proportion

* "A Manual for Observers in Climatology and Evaporation", by Frank H. Bigelow, U. S. D. A., Weather Bureau No. 409, p. 36.

Mr. Lee. to the area of the water surface. For this reason, the value of C_2 decreases with the increase in size of the area of water surface.

In passing, the writer draws attention to the fact that Professor Bigelow's coefficients are not for square pans, as the authors state, but for circular pans.*

(*b*)—*Relative Evaporation Depths from Land Pans and from Near-by Floating Pans.*—The writer's experience generally confirms the conclusion of the authors as set forth in Table 8. Thus, at the soil pan near Independence, the evaporation, from April, 1910, to March, 1911, was 85.06 in., as compared with 63.74 in. from a similar pan floating on the surface of Owens River not more than 2 miles away. The depth of evaporation from the floating pan was 75% of that from the soil pan. This relation is, to a certain extent, a special case of the preceding conclusion. The floating pan has practically the same evaporation rate as the large water surface, and the land pan has a greater rate, due partly to its being very much smaller than the lake, and partly to the higher day temperature of the water. The ratio, therefore, is not a fixed one, but varies with the size of the lake, the size of the land pan, and the difference in the water temperatures in the land pan and floating pan.

(*c*)—*Relative Evaporation Depth from Large Reservoirs, as Compared with That from 3-Ft. Square Pans Floating Thereon.*—The authors state (on page 1745†) that:

> "* * * it has been recognized more or less generally that the evaporation losses from large reservoirs * * * are materially less than the evaporation depths measured in pans, even when the pans are floating on the reservoir."

The writer does not agree with this statement, except in the case of land pans, as previously explained. In none of the quotations from Professor Bigelow is the statement made or inferred that the evaporation from floating pans differs materially from that of the water surface on which they are floating. The assumption by the authors (on page 1747†) that the values of C_2, as given by Professor Bigelow, are for floating pans, is not substantiated in any of the latter's writings. None of the pans used at Salton Sea was floating, and it was only there that Professor Bigelow had the opportunity to develop experimentally values for C_2, as published in 1910. The computation by the authors merely means that the evaporation from a large water surface is 61.7% of that from a 3-ft. circular (not square) pan outside of the vapor blanket.

On the same page the authors make the assumption that "the evaporation depths from pans 2 ft. above the surface of the [Salton]

* "Provisional Statement Regarding the Total Amount of Evaporation by Months at 23 Stations in the United States, 1909–10," by Professor F. H. Bigelow, U. S. D. A., Weather Bureau (Abstract of Data No. 4).

† *Proceedings*, Am. Soc. C. E., for September, 1915.

sea were the same as if the pans had been floating in the sea instead". The reverse of this statement occurs in numerous places in Professor Bigelow's writings. For example, in the second quotation on page 1746,* he states: Mr. Lee.

"If 70 ins. is admitted as the amount evaporated from Salton Sea, there remains 38 ins. as the difference between the [evaporation from the] water in the sea and that in the lower swinging pan [2 ft. above the surface of the sea]."

Again, he says, commenting on the work at Reno†:

"On the other hand there is a decided drop in the value of C_d, from 0.070 to 0.056, in passing from pan 3 to pan 1, that is to the surface of the water in the reservoir. It follows that the evaporation at the surface of the water body is $\frac{0.055}{0.070} = 0.79$ of that at a few feet above the surface. This same difference appears on the two-pan stands at Indio and Mecca, when the upper pan is evaporating 1.27 times faster than the one at the surface, after having applied the other functions of the formula."

The authors' unsupported assumption, therefore, is not in agreement with the exhaustive observations of Professor Bigelow, and hence the result of their computation near the bottom of page 1747* is seriously open to question.

The computations made by the authors on pages 1748-1749,* based on data of evaporation from Salton Sea as determined by F. T. Robson, Assoc. M. Am. Soc. C. E., and C. E. Grunsky, M. Am. Soc. C. E., from lake level and inflow, involve the assumption, already shown to be incorrect, that the relation of evaporation from floating pans of various sizes differs from that from the surface of the body of water in which they float, and vary as do land pans of similar size. The result of the computation, therefore, is without meaning.

The writer believes that this discussion shows the final conclusions of the authors, on pages 1750* and 1752,* to be fallacious, namely, that, in general, for large reservoirs, or at Lake Conchos, the yearly evaporation depth is 62% of the evaporation depth from a 3-ft. square pan floating thereon. On the other hand, he maintains that, with possibly a slight error, due to difference in temperature of the water, these evaporation depths are essentially the same.

(*d*)—*Relations Between Evaporation Depth and Mean Temperature.*—The authors' analysis of this subject is very interesting and suggestive, and the results are of value within the general region to which they can be applied and when evaporation records are not available. The writer places little confidence in the Piché evaporim-

* *Proceedings*, Am. Soc. C. E., for September, 1915.

† *Monthly Weather Review*, Annual Summary, 1908, pp. 437–445.

Mr. Lee. eter records used by the authors. He has had occasion, in several instances, to check them up with subsequent floating-pan records, and has found the results widely divergent. The authors have used Piché evaporimeter calculations at eleven of the seventeen points in New Mexico and Texas for which evaporation records are given in Table 17.

(e)—Relations Between Evaporation Depth and Elevation Above Sea.—The writer would make much the same comment on this section of the paper as he has on the preceding one. As of interest in connection with the subject, attention is called to the diagram, Fig. 9,* prepared by himself from observations by the Office of Experiment Stations, U. S. Department of Agriculture, showing the combined influences of altitude and temperature on evaporation on the east slope of Mt. Whitney, Inyo County, California.

The authors' principal conclusion will now be considered.

Method A.—The writer, as he has already explained at length, does not agree with the authors in the application of the correction factor of 62% to floating-pan evaporation depths, and therefore does not concur in their result of 52.5 in. as the annual evaporation at Lake Conchos. He does not mean to infer by this that he believes 84.74 in. represents the actual value. He does believe, however, that for all practical engineering purposes, floating-pan observations give the true depth of evaporation from a large body of water.

Methods B_a and B_b.—Both these methods involve the application of the factor, 62%, and, to that extent at least, the writer believes the results are in error.

Method C.—This method is the only one used by the authors which does not involve the correction factor, 62%, for floating pans. The results apparently agree closely with those obtained by the other methods. The writer does not place sufficient confidence in this method, however, to regard it as a check on the others. In the first place, the proportion of the annual evaporation depth assumed to occur during the 4 months, October to January, and the 7 months, October to April (Table 46), is based entirely on the application of temperature-evaporation studies in New Mexico and Texas to temperature conditions at Lake Conchos. Almost two-thirds of the New Mexico and Texas evaporation records, as previously shown, are Piché evaporimeter computations. Furthermore, the months under consideration are those of smallest evaporation loss, and a change of a few per cent. would make a great difference in the computed annual evaporation. For example, if the 4 months, October to January, were 17% instead of 21.7%, the result would be 71 in., instead of 55.4 in. At Independence, Cal., at Elevation 3 800, 500 ft. lower than Lake Conchos, the measured evaporation from a floating pan during these months is 14% of

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII, p. 186.

the annual.* Finally, a reservoir or lake, particularly one as recently flooded as Lake Conchos, does not show as great regularity of monthly variation in evaporation computed from inflow, outflow, and lake-level data, as pan observations. The writer has at hand accurate inflow and lake-level measurements at Owens Lake, California, for 8 years, and although the computed annual depths of evaporation agree closely, the monthly variation is very erratic, and agrees only in a very general way with a floating-pan record. Therefore, he does not believe that sufficient weight can be placed on Method *C* to regard it as supporting the other methods. Mr. Lee.

In concluding this discussion, the writer draws attention to what the authors state is the most important feature of their paper (page 1721†), namely,

"* * * it is estimated by a combination of the results of Methods *B* and *C* that at Lake Conchos the evaporation depth from the lake surface certainly is less than 67.5% (and in all probability as little as 62%) of that from a pan 3 ft. square floating thereon. In 1910 Professor Frank H. Bigelow, of the United States Weather Bureau (in connection with his Salton Sea evaporation experiments), deduced a value of 62% for this coefficient; and, so far as known to the writers, the foregoing value at Lake Conchos is the only check of Professor Bigelow's value which ever has been made."

The writer maintains, first, that the Lake Conchos observations did not prove, conclusively or even tentatively, that the evaporation from a large water surface differs appreciably from a properly installed pan floating thereon; second, that Professor Bigelow's work at Salton Sea did not prove, or even attempt to prove, that such a difference existed; and third, that, in these respects, the authors, throughout their paper, have labored under a misconception.

As positive proof of the writer's contention, he will state that the annual evaporation from a pan floating on the surface of Owens River, near Independence, Cal., was observed by him to be 67.2 in. averaged for two years;* that the annual evaporation from Owens Lake, which has no outlet, is 20 miles south of Independence, and has practically the same climatic conditions, is 61.0 in., as computed from accurate records of inflow and lake level for 8 years, together with a detailed area curve of the lake basin; and that exhaustive experiments conducted with fresh water and Owens Lake brine, the latter having an average specific gravity of 1.11, during the 8-year period, show that the evaporation rate from Owens Lake water at that specific gravity is 10% less than from fresh water. Hence, the evaporation from the floating-pan checks with that from Owens Lake to less than 1 per cent.

* *Water Supply Paper No. 294*, p. 118.

† *Proceedings*, Am. Soc. C. E., for September, 1915.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE AUTOMATIC VOLUMETER

Discussion.*

By George W. Booth, M. Am. Soc. C. E.

George W. Booth,† M. Am. Soc. C. E. (by letter).—This very ingenious and simple device should have a wide field of application, provided it proves in service to be as generally applicable to the various conditions of fluid flow as Mr. Hopson's tests lead him to believe. As he observes, one of the obstacles at present met in providing measuring devices of reasonable accuracy is the cost of putting in and maintaining the apparatus. Mr. Booth.

There appears to be no reason that this device cannot be used in measuring the flow in pipe distributing systems, provided the velocity of flow is sufficient to start the recording apparatus, as it usually will be in such systems; Mr. Hopson states that the velocity head required for this is as low as 0.001 ft.

The writer has been interested to determine whether the volumeter could be used on fire line services to buildings equipped with automatic sprinklers or standpipes. If it could be certain of use solely for fire protection, there would be no call for a meter, since this use occurs only at very infrequent intervals, and the total consumption is comparatively small; however, the experience of some water departments in connection with the illicit use of water for other purposes has been such that the placing of some form of meter is being required, and the cost of a satisfactory one is a considerable objection, especially in the smaller buildings.

* This discussion (of the paper by E. G. Hopson, M. Am. Soc. C. E., published in October, 1915, *Proceedings*, and presented at the meeting of December 1st, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

Mr. Booth. The most common size of fire line service is 6-in., with a 4-in. pipe for buildings having less than about 6 000 sq. ft. of floor area, and an occasional service of larger size. If the minimum velocity head at which the apparatus will register is 0.001 ft., corresponding to a velocity of about 0.3 ft. per sec., flows of less than about 22 gal. per min. in a 6-in. pipe, and about 10 gal. per min. in a 4-in. pipe, will pass unrecorded. A meter not capable of measuring these smaller flows would hardly be satisfactory for this service, and it would probably be necessary, as is the present practice with other types of meters, to provide for measuring them through a by-pass, with an arrangement to divert the flow to the main channel automatically at such times as a large supply and small loss of head are required.

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PAPERS AND DISCUSSIONS

THE CHERRY STREET BRIDGE, TOLEDO, OHIO
Discussion.*

By Messrs. Edward Godfrey and James Ritchie.

Edward Godfrey,† M. Am. Soc. C. E. (by letter).—This paper is another exemplification of the fact that engineers are capable of taking up and solving the big problems of construction, and for this reason it is a very valuable addition to engineering literature. Mr. Godfrey.

The paper also illustrates the fact that some of the smaller problems, the so-called details of design, are not given the thought and attention that their importance deserves. Engineering science is not balanced in this respect. The big things are worked out to the exclusion of details that are of equal, and sometimes of greater, importance.

It was a small thing that caused the Quebec Bridge to fall, a mere detail, the entire absence of bracing in a great traveler 217 ft. high. A great deal of work and thought were devoted to the subject of secondary stresses, which in material of the toughness of steel is of little or no consequence; but the simple matter of preventing the lateral swaying of 1 000 000 lb. of steel, held more than 200 ft. in the air, was given no consideration whatever.

The writer checked a set of details for structural work of two theaters. There were twenty-two types of errors, some of them repeated many times: Cantilevers had little or no provision for bending at the section of greatest moment; some had provision for tension but none for compression; curved girders had nothing to prevent them from flopping down, as did a large one in the Orpheum Theatre, in New York City, pulling the entire structure with it. Other errors were found. This was the work of one who held himself to be a capable structural engineer.

* This discussion (of the paper by Clement E. Chase, Jun. Am. Soc. C. E., published in October, 1915, *Proceedings*, and presented at the meeting of December 1st, 1915), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Pittsburgh, Pa.

Mr. Godfrey. With practically no exception, great structural failures (excluding dams) have been due to lack of bracing or lack of attention to details of design.

Fig. 3 shows that the columns of the approach viaduct of this bridge are of square and oblong shape, having upright rods and so-called hoops 12 in. apart. A rectangular hoop is the latest thing in "squaring the circle". The writer has publicly described and shown a photograph of a column of this kind that broke at 200 lb. per sq. in. due to the mere shock of a near-by wreck. These columns, by the writer's calculation, are loaded to 400 or 500 lb. per sq. in., and belong in the class that has characterized nearly all the many reinforced concrete wrecks, where the columns are broken up into short chunks or ground to a shapeless mass.

It is true that the Report of the Joint Committee allows columns of this sort, with only wires enough to hold the upright rods in place during the hardening of the concrete; it also allows plain concrete columns. If these columns were not more than 24 ft. high, and had no steel whatever, they would satisfy that Report, but the writer would hate to live under the viaduct.

The columns of this viaduct were not even chamfered on the corners. T. L. Condron, M. Am. Soc. C. E., says of the rodded columns of the Edison Building:*

"In places where the heat was not sufficient to destroy the insulation on electric wires strung through the ceiling, square columns were spalled almost as badly as those in the rest of the building where the heat was exceedingly great."

These facts and the failure of the column at 200 lb. per sq. in. indicate clearly that there is an intensity of strain in such columns ready to break out in failure from small contributing causes. All know what happened to the columns of the Edison Building, and how so many of them failed, and how so large a portion of one of the buildings collapsed on account of such failure, and also that such columns have been roundly condemned.

The rods in the Edison Building were 3 in. from the surface, and yet many of them broke out and split the columns from top to bottom, both those at the corners of columns and those in the middle of the sides. They did not have ties, but it is unthinkable that a few square hoops could redeem such members and make them fit to sustain a structure. The column which failed at 200 lb. per sq. in. was oblong, and had regulation ties, such as the oblong columns of this viaduct. The concrete was good and well seasoned.

The time will come when engineers must realize that not everything stated in books and committee reports is safe to follow, and the so-called

* *Journal*, Am. Concrete Inst., July, 1915, p. 397.

destructive critic must be reckoned with. The writer has been publicly condemning columns of this kind for ten years, and he has yet to hear any better argument in favor of them than this: "Mr. Godfrey's views are not in agreement with the majority of the Profession." Mr. Godfrey.

Circular or octagonal columns with upright rods (not for compression but for transverse strains) and close-spaced hooping constitute proper reinforced concrete.

Another criticism that the writer would make concerns the floor-beams of Fig. 3. The main reinforcing rods are bent up at 45 degrees. Some of them run to the support, and some stop short of it; but they are not anchored over the supports. The beams are cluttered with stirrups, the use of which no one has ever been able to explain. If these main reinforcing rods had been run up over the top of the support and then curved down for anchorage, or, in the case of the middle post, had extended well into the next girder for anchorage, the construction would be very greatly improved. In the case of a common support of two girders, there is a large tension in the top of the girder over the supports. The rods of one girder should be bent up and continued well into the other one, in order to take this tension and anchor those rods for the diagonal tension or shear. In this bridge, a number of long rods could have been left projecting out of one floor-beam to be built into the other when the latter is made. Steel disposed in this manner is of definite value, whereas the aimless stirrups commonly used are a hit-and-miss sort of makeshift.

The steel used in stirrups would be vastly more beneficial if it was converted into main reinforcing rods curved up and anchored over the supports. Tests and common sense demonstrate this. Besides increasing the strength and safety of structures, this would make them cheaper and simpler to build.

Another detail of design to which the writer would call attention is on Plate XLII. The sidewalk cantilever has a pull at the top flange which the writer calculates to be about 19 000 lb. To take this pull there are two rivets in tension on their heads, as shown by Plate XL. The angle to which the sidewalk bracket is attached is connected to the bascule girder by only two rivets, and the top chord of the bracket is pulling directly on their heads. Besides this, the connection is eccentric. This is a detail that is very commonly slighted, and illustrates what the writer stated at the beginning of this discussion. A detail worse than this one could scarcely be devised. It behooves the engineers of this bridge to see to the correction of it before the sidewalk drops into the river.

The writer is familiar with a long viaduct where the sidewalk brackets are dependent on tension on four rivet heads the severance of which would mean, not only the dropping of the sidewalk, but the wrecking of the supporting girder.

Mr. Ritchie. JAMES RITCHIE,* M. AM. SOC. C. E. (by letter).—The writer is much interested in this paper as he was in charge of the work for the firm of contractors which built all except the steel bascule spans. Mr. Chase has described very fully the methods used in the construction of the first half of the bridge, and a few additional details from the point of view of the contractor may be of interest.

Referring to the historical sketch, there are a few additional facts which came under the writer's observation. The first estimate of cost made by the designing engineers was $515 000, and on this basis the City of Toledo issued bonds and secured a fund of $525 000 for the building of the bridge. In the summer of 1907 proposals were asked for, but only one was received, which was more than $900 000. The matter was then discussed, and various plans were suggested, but finally, in July, 1909, proposals were again asked for, and three bids were received on the plans of the City and three alternate bids on the bidders' own plans. One of these alternate bids was for $897 000, and was made by the same firm which had bid alone on the first advertisement. One of the other bids was based on a different form of reinforcement of the arches from that shown on the City's plan, and the other bid was on a totally different method of construction. The lowest bid on the City's plan amounted to about $813 000, and therefore would require at least $300 000 more than the City had in its fund for the work. Even the lowest of the alternate plans required more than the City had appropriated, and the question which plan to adopt was discussed for several months. Finally, it was submitted to a board of expert engineers, and they recommended the adoption of the City's plan. The necessary additional funds were secured, and on March 3d, 1910, the contract was awarded. Immediately thereafter the firm which had submitted the lowest bid on an alternate design brought suit, through a resident of Toledo, to enjoin the City and the contractor from carrying out the contract. Decision on the suit was rendered in favor of the City and the contractor, in the Court of Common Pleas, and the case was appealed to the Circuit Court, which sustained the lower Court. Pending the decision in the Circuit Court, and at the request of the City, the contractor started work on the construction during June, 1910.

Plans were made by the writer which contemplated removing the old bridge to a temporary site and supporting the spans on pile piers, so as to leave the site of the new bridge unobstructed, but this idea was abandoned, not so much from fear of the action of ice on temporary work as because the recent decision of the Court had affirmed the fact that the method contemplated by the City's plans of building the superstructure in two halves was practicable, and the City did not wish to raise the question again before the public.

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* Cleveland, Ohio.

The easterly bascule pier, known as Pier IV, was placed so that the existing pier, supporting the east end of the 300-ft. swing bridge and the west end of a 175-ft. fixed span, had to be wholly removed and its foundation piles withdrawn before the bascule pier could be constructed. The coffer-dam for the bascule had to be at least 48 ft. 6 in. wide to contain the base of this pier, and this width had to be increased to 53 ft. on account of the projecting base of the old pier, which was at the extreme edge of the coffer-dam. In order to support the old bridge at this point without interfering with the river traffic, as well as the traffic over the bridge, it was necessary to have supporting trusses with a clear span of 60 ft., to provide very heavy pile supports for them, and to construct the trusses so that the old bridge would be supported rigidly for the fixed span and on the end wedges for the swing span. The temporary trusses had to sustain the loads at the site of the old pier, and this point was about 15 ft. from the west support and 45 ft. from the east support of the temporary trusses. Mr. Ritchie.

The old piers were first cut out by vertical slots of the width necessary to receive the temporary trusses, leaving a stone column to support the ends of the trusses of the old bridge. The temporary trusses were then placed in position and, after they were ready, the stone column was cut out for a sufficient space in which to place a jack on each side of the end post to hold it, while enough stone was removed to permit the placing of the wooden beams which were to transfer the load to the temporary trusses. This latter work had to be performed during the night when the traffic could be interrupted, and this was only from midnight until 5 A. M.

Four timber trusses were used at the bascule span and four plate girders at the east abutment. The top and bottom chords of the Howe trusses were each made of two pieces of 12 by 16-in. Oregon fir, each piece being 84 ft. long, thus being the full length of the trusses over the supports and requiring no splices, and thereby requiring only enough falsework to keep the sag out of the timbers while putting in the web members. The plate girders at the east abutment were put in position by a floating derrick, and also required no falsework for their erection.

The old bridge had a total width of 42 ft., and, in order to construct the north half of the new bridge, it was necessary to remove the north sidewalk of the old bridge. It was also necessary to move the two end spans into line with the other spans. These were originally placed so that there was a bend in the roadway on the bridge. To make this change in the end spans, falsework was built at the south ends of the piers, and, when all was ready, the two spans were jacked over to their new position on a Sunday afternoon.

Mr. Ritchie. While the first half of the bridge was being constructed, the City awarded a contract for a new foundation of a bridge at Ash Street, about a mile north of Cherry Street, contemplating the use of the old Cherry Street spans for the superstructure of the Ash Street Bridge, and as soon as traffic was turned over the first half of the new Cherry Street Bridge, the old spans were taken down, new floor-beams ordered, necessary repairs made to the old parts, and the spans erected on the Ash Street piers. The old trusses were found to be in good condition, and, being of wrought iron, the members had not deteriorated in the 30 years of use, having been kept well painted during all that period.

Mr. Chase has well said that the removal of the old bridge and the experience gained by the company in constructing the first half, was the reason for the rapidity with which the second half was completed. The company regrets that the plan of removing the old bridge to a temporary site—suggested in the first place—was not carried out, as experience has shown that the cost of such removal would have been infinitesimal compared with the expense incurred and the delays occasioned by having to construct the bridge in two parts. The company's records show that the whole bridge could have been completed by the time that the first half was ready for traffic, thus saving more than a year of overhead expense, to say nothing of such difficulties as the obstruction caused by the piers, the temporary supports, the building of two coffer-dams, instead of one, for six of the piers, and the maintenance of traffic over the old bridge.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

PROGRESS REPORT OF THE SPECIAL COMMITTEE ON MATERIALS FOR ROAD CONSTRUCTION AND ON STANDARDS FOR THEIR TEST AND USE*

To the American Society of Civil Engineers,

Gentlemen:

Your Special Committee on "Materials for Road Construction, and on Standards for Their Test and Use", has, during the past year, endeavored mainly to consider the new field assigned to it by the Board of Direction at the Annual Convention in June, 1914, and to prepare for presentation, at this time, such a report on materials other than bituminous used in road construction as would thereby place both the bituminous and the non-bituminous highway materials on the same footing, in so far as the work of this Committee and its duties to the Society are concerned.

The demands on the time of this Committee have been so great as to prevent a satisfactory revision of the Semi-Final Report on Bituminous Materials submitted in 1915 and to compel it to defer such revision for another year. It is the intention of your Committee during 1916 to review its reports of 1915 and 1916 and to present a revised report on both these branches of its subject to the Annual Meeting in January, 1917.

SEMI-FINAL REPORT ON NON-BITUMINOUS MATERIALS.

Your Committee first wishes to call attention to the propriety of the application to non-bituminous materials of many of the conclusions submitted in its Semi-final Report on Bituminous Materials used in highway work, and, in order to avoid unnecessary repetition thereof in this report, it recommends reference to its report of a year ago,

* To be presented to the Annual Meeting, January 19th, 1916.

and the application of all its conclusions, therein expressed, as may be proper in this connection, to non-bituminous highway materials, in addition to the specific statements hereinafter made.

A considerable variety of non-bituminous materials and of methods of using them exists in highway construction and maintenance. Some of these materials have been in use for many years, and some of the practice in the test or use of them has become fairly well standardized. On the other hand, many of the materials are new, and more or less change has occurred in their tests or use, and is even now taking place. Local conditions have had much to do with the selection of the material and the determination of the particular method to be followed in any specific case.

Your Committee has considered it impracticable to attempt to cover all the various non-bituminous road materials in this report, but has considered especially what seem to be the more important ones. It believes that many of the questions affecting the selection or use of these materials can only be solved by a better knowledge of their characteristics or qualities and more complete records along uniform lines of their behavior in use. It, therefore, recommends that the needed information concerning the materials be secured along the lines laid down under the various headings, in order that conclusions of value may be reached and a general agreement in the records thus obtained.

As in its work on bituminous materials, your Committee has attempted to initiate and encourage the collection of comparable records concerning non-bituminous highway materials by arranging, for the purpose, forms which will be widely distributed to highway authorities. Copies of these forms will be found in Appendices A and B.

The Committee, recognizing the importance of the factor of costs, both first and subsequent, in almost all pavement questions, has, for the same reasons, attempted to initiate and encourage the collection of comparable records of cost figures by the preparation of forms for this purpose. Copies of these forms will be found in Appendix A. Both these forms apply to the particular material or methods used on specific areas of road or street surfaces. For comparisons of cost figures in connection with the study of relative costs between streets or considerable sections of a traffic route, your Committee reiterates the recommendations contained in its report of January 20th, 1915, under the heading, "Collection and Standardization of Cost Data". It should also be noted that the traffic data provided for in the part of Appendix A relating to cost data are not intended to supplant those given under traffic census.

In submitting its conclusions, your Committee has, for convenience, arranged them under separate classifications. It should be distinctly

understood that the Committee has deliberately refrained from including in this report such conclusions regarding any material or method as appear to have been generally agreed on. On the other hand, no conclusions have been stated unless supported by a substantial majority of the Committee's membership.

Your Committee offers such conclusions (other than its forms referred to) as it has been able to reach during the past year on non-bituminous highway materials as follows:

General.—Your Committee considers it still of sufficient importance to warrant the repetition here of its previous statements to the effect that there is no such thing as "the most satisfactory roadway surface" or as a "panacea" for all highway ills, and that the proper selection of the particular material and methods of construction to be used, which will most efficiently meet the conditions of any particular case, is the real problem to be solved.

Your Committee recommends that the selection of the kind of crust or pavement be based on the following factors, the special value of which may be estimated in each case under the local conditions of traffic, surroundings, climatic conditions, and physical and financial resources, both as to construction and maintenance, with proper regard for probable or possible changes in these circumstances: First cost, maintenance cost, annual cost (interest on first cost plus maintenance cost plus annuity), ease of maintenance (facility of making repairs), durability, cleanliness, tractive resistance, slipperiness, favorableness to travel, sanitariness (especially when not properly cleaned), noiselessness, and appearance.

Broken Stone and Slag Roadways.—The size of stone in the wearing course should be as large as practicable, but not so large as to lead to its being dislodged under traffic.

Within limits, the softer or more cementitious the stone used for the wearing course the larger its size should be.

The size of stone in the bottom course should be within the maximum limit which will insure its stability under traffic and other stresses, and may be of any size that will result in the economical use of the crusher product.

The utmost possible compaction of the courses, before the addition of the void-filling material, is desirable.

Thorough filling of the otherwise irreducible voids is desirable, and a slight excess of the void filler on the surface of the wearing course will usually be found to be of value.

For a water-bound broken-stone roadway, the void filler should be clean stone screenings or sand, and the use of clay should be avoided. A proportion of fine mineral material, between certain maximum and minimum sizes, which proportion and sizes will depend largely on the character of the materials used, is desirable.

For water-bound roadways of some limestone or of some slags, it will be advisable wherever practicable to use for one-half the void filler a non-cementitious material such as clean sand with the other half of the finer particles of the material itself.

Dense, tough slag is preferable to the more porous or to the vitreous varieties, and it should be free from metallic pieces.

Such proportions of the various sizes of material used as will result in the greatest possible density of the roadway, when properly compacted and bound, are desirable.

Gravel Roadways.—The conclusions expressed under the head of "Broken Stone and Slag Roadways" apply in a general way to gravel roadways, except as follows:

With rounded gravel the tendency toward dislodgment under traffic is greater than with angular broken stone, and hence, for equal resistance to this tendency, the size of the pieces in the courses of the roadway must be somewhat smaller with gravel than in the case of broken stone.

With gravels such as quartz, the cementation of which is extremely low, a highly cementitious void filler is desirable, and a moderate quantity of clay in sand used for filling the interstices of water-bound gravel surfacing may be advantageous where the water and frost action on the roadway surfacing is not too severe.

Where the roadway is constructed of run-of-bank gravel, it is desirable to obtain a gravel which will contain enough stone of the larger size to insure stability and wearing qualities under traffic, and which contains sufficient finer material to insure a proper bond.

The Committee is of the opinion that uncarpeted roadways should be used only for light traffic.

Cement-Concrete Pavements.—The sub-grade for a cement-concrete pavement should always be as carefully prepared, rolled, and compacted as for any other roadway, and should be made to conform to the proper lines and grades.

The value of a sand layer under cement-concrete pavements, to allow for expansion and contraction, appears as yet to be undetermined, and further experimental data bearing on this point are desired.

Expansion joints, when provided in a roadway slab or pavement, should be designed and installed so as to interrupt to the minimum degree practicable the uniformity of the surface; should be placed at intervals of approximately 30 ft.; and may be built with advantage at an angle of from 70 to 80° with the axis of the road.

Too much emphasis cannot be laid on the importance of frequent testing of the materials. This testing should include, not only the cement, but also the fine and coarse aggregates. The most desirable fine aggregate is that which is properly graded from the maximum-sized particles downward, so as to have the greatest density.

The importance of thoroughly mixing the concrete is of such great moment that it can hardly be over-emphasized. The denser the concrete, other things being equal, the greater will be its strength and value. Every effort should be made to obtain by proper proportion and workmanship the greatest density practicable, under the existing conditions of any case, and the proportions of the various ingredients should be rationally determined and not be arbitrarily dictated or assumed.

Uniformity of character and composition being a most desirable quality of the completed pavement, every effort must be made to meet and offset the various tendencies toward non-uniformity particularly prevalent in concrete work, such as variations in proportions of sizes and materials, segregation of the mix during its preparation, and variation in compaction, setting, and drying of the mass.

In all cases the surface of the finished pavement should be kept wet and, if possible, protected from the sun for several days.

Brick and Slag Block Pavements.—Toughness, resistance to wear from shock or abrasion, and non-absorption are essential qualities. The first two can be determined by the standard rattler test, and the last by the customary absorption test through immersion in water. Uniformity in the rate of wear is equally important, and may properly be a controlling consideration, even at the expense of a moderate sacrifice in the rate of wear.

Although the use of sand joints may occasionally appear to be justified, in the interest of economy, it is unwise to use them when some additional first cost will result in appreciably prolonging the life of the pavement. Cement joints, when properly made, will maintain the integrity of the surface, but uniformity in the cement grout and special skill and care in its application are essential to success.

Such form of the individual brick or block is desirable as will automatically provide sufficient, but not too wide, joints, and as will insure uniformity in their width, even when they are laid rapidly and with ordinary care.

The thickness of the sand cushion often used is excessive. The function of this cushion is to give some resiliency to the wearing course, and to allow for irregularities in the surface of the concrete foundation and for unavoidable variations in the depth of the brick or block. As the variation in depth (of the brick or block) decreases, the thickness of the sand cushion can be correspondingly decreased. If the variation in the depth does not exceed $\frac{1}{8}$ in. and if the surface of the concrete foundation is sufficiently even, a cushion course 1 in., or even $\frac{3}{4}$ in., in depth will be enough.

The Committee recommends further investigation of the possible use of a bituminous cushion and a cement mortar bed in place of the sand cushion for brick and slag block pavements.

Stone Block Pavements.—The recent and present tendencies toward smaller and better dressed blocks should be encouraged, to the end that closer joints between the blocks may be had and that greater uniformity in the thickness of the cushion under the blocks may be secured. Under average conditions, for a given class of pavement, stone blocks used in various parts of America should, in the interests of low first cost and economical manufacture, comply with the same requirements relative to dimensions and dressing of the several faces.

The thickness of the sand cushion often used is excessive. The function of this cushion is to give some resiliency to the wearing course, and to allow for irregularities in the surface of the concrete foundation and for unavoidable variations in the depth of the block. As the variation in depth (of the block) decreases, the thickness of the sand cushion can be correspondingly decreased. If the variation in the depth does not exceed ½ in., and if the surface of the concrete foundation is sufficiently even, a cushion course 1 in. in depth will be enough. The Committee recommends further investigation of the possible use of a bituminous cushion in place of the sand cushion for stone block pavements.

Stone block pavements should generally be laid on a cement-concrete foundation, but, in case of temporary paving, any type of well-drained, stable foundation may be used.

Stone blocks should be composed of medium or fine-grained granite or sandstone having a percentage of wear of not more than 4.5, and a toughness of not less than 8. Sandstone and granite for stone blocks should have crushing strengths of not less than 16 000 and 20 000 lb. per sq. in., respectively.

As it is desirable to secure a suitable water-proof wearing course for pavements, sand should never be used alone as the joint filler. A bituminous filler may be preferred to a cement grout filler on account of the lower cost of street-opening repairs, the better foothold provided for horses, and the securing of a more resilient and, hence, less noisy pavement.

The utilization of recut stone blocks has been demonstrated to be capable of producing economical and satisfactory results, and should be encouraged.

Wood Block Pavements.—There is no general necessity for confining the material for wood paving blocks to long-leaf Georgia pine (*Pinus palustrus*).

Such form and shape of the individual block, or such method of laying as will insure the placing of the blocks with the fiber normal to the surface of the pavement and provide uniform joints, even when the blocks are rapidly and not over-carefully placed, are desirable.

Experience has demonstrated that an excessive quantity of preservative involves additional cost without compensating advantages, and, if too little is used, it may fail of its purpose. Under normal conditions, it would appear that not less than 15 nor more than 20 lb. per cu. ft. should be used.

No necessity exists, in the case of a wood block pavement, for a resilient cushion under the blocks, and the reduction of any provision for a cushion or bed, in or on which the blocks may be laid, to such a layer as may be necessary safely for the purpose of correcting the unevennesses of the foundation and of permitting the immediate compensation for irregularities in the depths of the blocks themselves, is desirable. It is also desirable that such an intermediate layer between the foundation and the blocks shall be of such character as will insure its permanence in the position it occupies when the pavement is opened to traffic.

In filling joints, three different materials have been used, and all have their advocates. Although it is possible that all are good under different circumstances, neither experience nor experiments has made it certain which one is the best.

Your Committee wishes to take this opportunity to express its appreciation of the interest shown in its previous report, and of assistance given the Committee by those who contributed discussion on it. It regrets that, as stated, pressure of other matters has prevented proper consideration of the criticisms or discussion offered, up to this time, but, during the coming year, it hopes to be able to digest properly the suggestions made, and thereupon to embody revised and further conclusions in its next report.

The Committee wishes to express again its deep appreciation of the assistance given it by the Board of Direction, and by the Secretary, Mr. Charles Warren Hunt, as well as by the members of the Society and others.

Very respectfully,

Special Committee on Materials for Road Construction, Etc.

W. W. Crosby, *Chairman,*
H. K. Bishop,
A. W. Dean,
N. P. Lewis,
C. J. Tilden,
G. W. Tillson,
A. H. Blanchard, *Secretary.*

APPENDIX A

FORM OF RECORD FOR DATA CONCERNING THE USE OF NON-BITUMINOUS HIGHWAY MATERIALS

Data Concerning the Use of Non-Bituminous Highway Materials.

General Information.

State..........................County ....
Town..........................Road name ....
Limits of improvement...
Length of improvement, in feet.....................................
Width of crust or pavement, in feet (average)......................
Area of crust or pavement, in square yards.........................
Percentage of grade (maximum) length.........feet........per cent.
" " " (minimum) " .... " .... "
" " " (mean) " .... " .... "
Amount of crown, maximum..............minimum...............
Date of beginning and completion of improvement........to........
Hours of working day............Labor wage per hour............
Nature of sub-grade..
Maximum and minimum air temperature during year................

Class of Highway or Nature of Traffic.

Amount of Traffic or Traffic Census for........Hours (Average).
Dates of census...

	Commercial.		Estimate (in pounds) of maximum load per inch of tire.	Passenger vehicles.
	Empty.	Loaded.		
One-horse vehicle...............				
Two or three-horse vehicles......				
Four or more horse vehicles......				
....				
Motor cycles.....................				
" runabouts				
" touring cars (open or closed).				
" busses..................				
" trucks..................				

Remarks.—The reporter is requested to note under Remarks (near the end of this form), wherever possible, any special characteristics of the traffic, more particularly a preponderance of any one kind of traffic, speed, and wheel diameters, the Committee believing that these factors may affect the actual cost of maintenance.

Form of Construction.

(*i. e.*, gravel, broken stone, brick, stone block, wood block, and cement-concrete, and on what type of foundation.)

Construction and Cost Details.

A.—Foundation.

1. Material ....
2. Thickness ....
3. Cost per square yard..
4. Estimated life ...... years.

B.—Wearing Course.

1. Material ....
2. { Size of blocks..
   Thickness ....
3. Kind of joints..
4. Proportions of aggregate (cement-concrete)......................
5. Cushion or bed..
6. First cost per square yard....................................
7. Life ...... years.
8. Average annual maintenance cost per square yard during life of wearing course ....

C.—Traffic Data.

1. Tons per year (2 000 lb.)......................................
2. Average tons per yard of width..............................
3. Proportion of tonnage on metal tires..........................
4. " " C-3 on tires 2 in. or less in width..................

Items of Cost for Each Square Yard.

	Foundation	Wearing Course.
Materials		
Labor		
Superintendence		
Overhead, including interest on plant, depreciation, etc.		

NOT TO BE FILLED IN BY REPORTER.

DATA PER SQUARE YARD.

A-3.—First cost of foundation..............................
A-5.—Annual interest and sinking fund for foundation........
A-6.—Total annual cost of foundation........................
B-8.—Annual maintenance cost of wearing course..............
B-9.—Annual interest and sinking fund for wearing course......
B-10.—Total annual cost of wearing course...................

Yearly cost per 1 000 tons of traffic $= \frac{A\text{-}6 + B\text{-}10}{C\text{-}2}\ 1\,000 =$ ......

NON-BITUMINOUS HIGHWAY MATERIALS.

TESTS AND ANALYSES.

Broken Stone and Broken Slag.

Name and origin........................	....
Specific gravity	....
Absorption of water per cubic foot.......	....
Abrasion, percentage of loss.............	....
Toughness	....
Cementation	....
Crushing strength per square inch........	....
Mechanical analysis	(Use table at end of form)
Voids, percentage of, loose and compacted.	....

Gravel.

Location	....
Specific gravity	....
Abrasion, percentage of loss.............	....
Cementation	....
Mechanical analysis....................	(Use table at end of form)
Voids, percentage of, loose and compacted.	....

Sand.

Location	....
Specific gravity	....
Mechanical analysis....................	(Use table at end of form)
Voids, percentage of, loose and compacted.	....
Tensile strength in cement briquettes, as compared with standard Ottawa sand..	....

Mixtures of Sand or Other Fine Highway Materials
With Broken Stone, Broken Slag, or Gravel.

Specific gravity	....
Mechanical analysis	(Use table at end of form)
Voids, percentage of, loose and compacted.	....

Paving Brick.

Composition	....
Name of manufacturer..................	....
Rattler test, percentage of loss..........	....

Stone Block.

Name and origin.......................	....
Specific gravity.......................	....
Absorption of water per cubic foot......	....
Abrasion, percentage of loss............	....
Toughness	....
Hardness	....
Crushing strength per square inch......	....

Wood Block.

Character of wood......................	....
Weight of blocks......................	....
Soundness	....
Rings per radial inch..................	....
Quantity of preservative...............	....
Absorption of water after treatment......	....
Character of preservative:	
Specific gravity at 25° cent..........	....
Specific gravity at 38° cent..........	....
Solubility in benzol or chloroform.....	....
Water content.......................	....
Distillation:	
Up to 170° cent..................	....
170° to 200° cent................	....
200° to 210° cent................	....
210° to 235° cent................	....
235° to 270° cent................	....
270° to 300° cent................	....
300° to 315° cent................	....
315° to 355° cent................	....

Mechanical Analysis.

		Percentages by weight.		
Passing 200-mesh sieve,	....	.....	.....	.....
" 100 " "	retained on 200-mesh sieve,	.....	.....	.....
" 80 " "	" " 100 " "	.....	.....	.....
" 50 " "	" " 80 " "	.....	.....	.....
" 40 " "	" " 50 " "	.....	.....	.....
" 30 " "	" " 40 " "	.....	.....	.....
" 20 " "	" " 30 " "	.....	.....	.....
" 10 " "	" " 20 " "	.....	.....	.....
" ¼-in. screen,	" " 10 " "	.....	.....	.....
" ½ " "	" " ¼-in. screen,	.....	.....	.....
" ¾ " "	" " ½ " "	.....	.....	.....
" 1 " "	" " ¾ " "	.....	.....	.....
" 1¼ " "	" " 1 " "	.....	.....	.....
" 1½ " "	" " 1¼ " "	.....	.....	.....
" 2 " "	" " 1½ " "	.....	.....	.....
" 2½ " "	" " 2 " "	.....	.....	.....
" 3 " "	" " 2½ " "	.....	.....	.....
" 3½ " "	" " 3 " "	.....	.....	.....
		.....	.....	.....

Remarks..

Name ....
Title ....
Address ....
....

Date .... 191... *Reporting Officer.*

APPENDIX B

ANALYSES AND TESTS OF NON-BITUMINOUS MATERIALS

The adoption of the following lists of tests, as including all those probably of value in determining and recording the characteristics of non-bituminous highway materials, is recommended.

Broken Stone and Broken Slag.

Name and origin..
Specific gravity..
Absorption of water per cubic foot.......................
Abrasion test, percentage of loss........................
Toughness test..
Cementation test..
Crushing strength per square inch........................
Mechanical analysis.......................................
Voids, percentage of, loose and compacted.................

Gravel.

Location ....
Specific gravity..
Abrasion test, percentage of loss........................
Cementation test..
Mechanical analysis.......................................
Voids, percentage of, loose and compacted.................

Sand.

Location ....
Specific gravity..
Mechanical analysis.......................................
Voids, percentage of, loose and compacted.................
Tensile strength in cement briquettes, as compared with standard Ottawa sand..

Mixtures of Sand or Other Fine Highway Materials with Broken Stone, Broken Slag, or Gravel.

Specific gravity..
Mechanical analysis.......................................
Voids, percentage of, loose and compacted.................

Paving Brick.

Composition ....
Name of manufacturer......................................
Rattler test...

Stone Block.

Name and origin....................................

Specific gravity....................................

Absorption of water per cubic foot....................

Abrasion test, percentage of loss.....................

Toughness test.......................................

Hardness test..

Crushing strength per square inch......................

Wood Block.

Character of wood...................................

Weight of blocks per cubic foot........................

Soundness ....

Rings per radial inch................................

Quantity of preservative per cubic foot.................

Absorption of water after treatment....................

Character of preservative:

Specific gravity at 25° cent. (77° Fahr.) (See *Proceedings*, Am. Soc. C. E., for December, 1914, p. 3036)..

Specific gravity at 38° cent. (100° Fahr.).............

Solubility in benzol or chloroform.................

Water content.....................................

Distillation:

Up to 170° cent. (338° Fahr.)....................

170° cent. (338° Fahr.) to 200° cent. (392° Fahr.)...

200° " (392° ") to 210° " (410° ")...

210° " (410° ") to 235° " (455° ")...

235° " (455° ") to 270° " (518° ")...

270° " (518° ") to 300° " (572° ")...

300° " (572° ") to 315° " (599° ")...

315° " (599° ") to 355° " (671° ")...

Cement.

All properties determined in accordance with methods recommended by the Society's Special Committee on Concrete and Reinforced Concrete.*

It is further recommended that the following methods for performing these tests be adopted as standards:

Apparent Specific Gravity of Rock.†

"The apparent specific gravity of rock shall be determined by the following method: First, a sample weighing between 29 and 31 g. and approximately cubical in shape shall be dried in a closed oven for

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, pp. 427–430.

† Proposed in 1914 by Committee D–4, "Standard Tests for Road Materials", of the Am. Soc. for Testing Materials.

1 hour at a temperature of 110° C. (230° F.) and then cooled in a desiccator for 1 hour; second, the sample shall be rapidly weighed in air; third, trial weighings in air and in water of another sample of approximately the same size shall be made in order to determine the approximate loss in weight on immersion; fourth, after the balances shall have been set at the calculated weight, the first sample shall be weighed as quickly as practicable in distilled water having a temperature of 25° C. (77° F.); fifth, the apparent specific gravity of the sample shall be calculated by the following formula:

$$\text{Apparent specific gravity} = \frac{W}{W - W_1}$$

in which W = the weight in grams of the sample in air and W_1 = the weight in grams of the sample in water just after immersion.

"Finally, the apparent specific gravity of the rock shall be the average of three determinations, made on three different samples according to the method above described."

Apparent Specific Gravity of Sand, Stone Screenings, or Other Fine Highway Material.

Apparatus.—The determination shall be made with a Jackson specific gravity apparatus which shall consist of a burette, with graduations reading to 0.01 in specific gravity, about 23 cm. (9 in.) long and with an inside diameter of about 0.6 cm. (0.25 in.), which shall be connected with a glass bulb approximately 13 cm. (5.5 in.) long and 4.5 cm. (1.75 in.) in diameter, the glass bulb being of such size that from a mark on the neck at the top to a mark on the burette just below the bulb, the capacity is exactly 180 c.c. (6.09 oz.); and an Erlenmeyer flask which shall contain a hollow ground-glass stopper having the neck of the same bore as the burette and a capacity of exactly 200 c.c. (6.76 oz.) up to the graduation on the neck of the stopper.

Method of Determination.—The method shall consist of: First, dry at not more than 110° cent. (230° Fahr.) to a constant weight a sample weighing about 55 grammes; second, weigh to 0.1 gramme, 50 grammes of the dry sample and pour it into the unstoppered Erlenmeyer flask; third, fill the bulb and burette with kerosene, leaving just space enough to take the temperature by introducing a thermometer through the neck; fourth, remove the thermometer and add sufficient kerosene to fill exactly to the mark on the neck, drawing off any excess with the burette; fifth, run into the flask about one-half of the kerosene in the bulb to remove air bubbles and then run in more kerosene, removing any material adhering to the neck of the flask, until the kerosene is just below the ground glass; sixth, place the hollow ground-glass stopper in position and turn it to fit tightly, and then run in kerosene exactly to the 200-c.c. (6.76-oz.) graduation on the neck, care being taken to remove all air bubbles in the flask; seventh, read the specific gravity

from the graduation on the burette, and the temperature of the oil in the flask, noting the difference between the temperature of the oil in the bulb before the determination and that of the oil in the flask after the determination; eighth, make a temperature correction to the reading of the specific gravity in accordance with the table furnished by the manufacturer of the apparatus, adding the correction if the temperature of the kerosene has increased and subtracting it if the temperature of the kerosene has decreased.

Absorption of Water Per Cubic Foot of Rock.*

"The absorption of water per cubic foot of rock shall be determined by the following method: First, a sample weighing between 29 and 31 g. and approximately cubical in shape shall be dried in a closed oven for 1 hour at a temperature of 110° C. (230° F.) and then cooled in a desiccator for 1 hour; second, the sample shall be rapidly weighed in air; third, trial weighings in air and in water of another sample of approximately the same size shall be made in order to determine the approximate loss in weight on immersion; fourth, after the balances shall have been set at the calculated weight, the first sample shall be weighed as quickly as possible in distilled water having a temperature of 25° C. (77° F.); fifth, allow the sample to remain 48 hours in distilled water maintained as nearly as practicable at 25° C. (77° F.) at the termination of which time bring the water to exactly this temperature and weigh the sample while immersed in it; sixth, the number of pounds of water absorbed per cubic foot of the sample shall be calculated by the following formula:

$$\text{Pounds of water absorbed per cubic foot} = \frac{W_2 - W_1}{W - W_1} \times 62.24$$

in which W = the weight in grams of sample in air, W_1 = the weight in grams of sample in water just after immersion, W_2 = the weight in grams of sample in water after 48 hours immersion, and 62.24 = the weight in pounds of a cubic foot of distilled water having a temperature of 25° C. (77° F.).

"Finally, the absorption of water per cubic foot of the rock, in pounds, shall be the average of three determinations made on three different samples according to the method above described."

Abrasion Test for Broken Stone or Broken Slag.†

"The machine shall consist of one or more hollow iron cylinders; closed at one end and furnished with a tightly fitting iron cover at the other; the cylinders to be 20 cm. [7.87 in.] in diameter and 34 cm. [13.38 in.] in depth, inside. These cylinders are to be mounted on a shaft at an angle of 30° with the axis of rotation of the shaft.

"At least [13.6 kg.] 30 lbs. of coarsely broken stone shall be available for a test. The rock to be tested shall be broken in pieces as nearly uniform in size as possible, and as nearly 50 pieces as possible shall constitute a test sample. The total weight of rock in a test

* Proposed in 1914 by Committee D-4, "Standard Tests for Road Materials", of the Am. Soc. for Testing Materials.

† Method adopted by the Am. Soc. for Testing Materials, August 15th, 1908.

shall be within 10 grams of 5 kilograms [11.02 lb.]. All test pieces shall be washed and thoroughly dried before weighing. 10 000 revolutions, at the rate of between 30 and 33 to the minute, must constitute a test. Only the percentage of material worn off which will pass through a 0.16 cm. (1-16 inch) mesh sieve shall be considered in determining the amount of wear. * * *"

Abrasion Test for Gravel.

The test for abrasion of gravel shall be made with a Deval abrasion machine. (See "Abrasion Test for Broken Stone or Broken Slag".)

A charge of gravel shall consist of pieces which shall pass a screen having circular openings 5.08 cm. (2 in.) in diameter and be retained on a screen having circular openings 1.27 cm. (½ in.) in diameter. The total weight of gravel in a charge shall be within 10 grammes of 5 kg. (11.02 lb.). The gravel to compose a charge shall be washed, and dried in a closed oven for 1 hour at a temperature within 5° of 110° cent. (230° Fahr.). The charge of gravel shall be placed in one cylinder of the machine, which shall be rotated at a rate of not less than 30 nor more than 33 rev. per min. Ten thousand revolutions shall constitute a test. The percentage of material worn off which will pass through a sieve having openings of 0.16 cm. ($\frac{1}{16}$ in.) shall be considered the amount of wear of the charge of gravel. The loss by abrasion, determined as stated, shall be expressed in terms of the percentage of the total weight of the charge of gravel.

Toughness Test for Rock or Slag.*

"1. Test pieces may be either cylinders or cubes, 25 mm. [0.98 in.] in diameter, and 25 mm. [0.98 in.] in height, cut perpendicular to the cleavage of the rock. Cylinders are recommended as they are cheaper and more easily made.

"2. The testing machine shall consist of an anvil of 50 kgs. [110.23 lb.] weight, and placed on a concrete foundation. The hammer shall be of 2 kgs. [4.41 lb.] weight, and dropped upon an intervening plunger of 1 kg. [2.2 lb.] weight, which rests on the test piece. The lower or bearing surface of this plunger shall be of spherical shape having a radius of 1 cm. [0.39 in.]. This plunger shall be made of hardened steel, and pressed firmly upon the test piece by suitable springs. The test piece shall be adjusted, so that the center of its upper surface is tangent to the spherical end of the plunger.

"3. The test shall consist of a 1-cm. [0.39-in.] fall of the hammer for the first blow, and an increased fall of 1 cm. [0.39 in.] for each succeeding blow until failure of the test piece occurs. The number of blows necessary to destroy the test piece is used to represent the toughness, or the centimeter-grams of energy applied may be used."

Hardness Test for Rock or Slag.

The test for hardness shall be made with a "Dorry", or similar machine, consisting of a revolving disk on which is fed, at a uniform

* Method adopted by the Am. Soc. for Testing Materials, August 15th, 1908.

rate, a standard quartz sand passing a 30- and retained on a 40-mesh sieve. Two cores, each 25 mm. (0.98 in.) in diameter, shall be cut from the material to be tested, and their faces ground off so as to be at right angles to the long axes of the cores. The cores shall be placed in the holders or dies and weighted so that the entire weight of each core with its holder and added weight is 1 250 grammes. Each core shall be ground in the machine on one face for 1 000 revolutions, after which it shall be reversed and ground on the other face for an equal number of revolutions. The loss of weight of each specimen shall be determined at the end of each 1 000 revolutions, and the average loss in weight shall be used for stating the hardness of the material, which latter shall be expressed by the formula: Hardness $= 20 - \frac{1}{3} W$, where W equals the average loss in grammes per 1 000 revolutions.

Cementation of Rock, Slag, and Gravel Powders.

The cementation test shall be made as follows: Of the material to be tested, 500 grammes shall be broken to pass a 1.27-cm. (½-in.) mesh sieve and then placed in a ball mill with 90 c.c. (3.04 oz.) of water and two steel shot weighing together 9 kg. (20 lb.). The mill and its charge shall be revolved for 2½ hours at a rate of 2 000 revolutions per hour. The dough thus formed shall then be removed, and 25 grammes of an average sample of it shall be placed in a metal die, 25 mm. (0.98 in.) in diameter, and subjected to a pressure of 132 kg. per sq. cm. for an instant in a hydraulic press. The cylindrical briquette resulting should measure exactly 25 mm. (0.98 in.) in height. If it does not, subsequent samples of the dough shall be taken in such quantity that the resulting briquette after compression will be exactly 25 mm. (0.98 in.) in height. Five such briquettes shall be made and allowed to dry in the air for a period of 20 hours, after which they shall be heated for 4 hours in a hot-air oven at a temperature of 93.3° cent. (200° Fahr.), and then cooled in a desiccator for 20 min. These cylinders or briquettes shall then be tested in a machine as follows:

The machine shall be arranged so that a 1-kg. (2.20-lb.) hammer is raised to a height of 1 cm. (0.39 in.), and then falls freely on a plunger transmitting the shock of the blows of the hammer through the plunger to the test piece, successive blows being struck by the hammer at a rate of 40 to 70 per min., until the test piece fails, which is indicated by the failure of the plunger or hammer to rebound. The test piece shall be placed on the anvil under the plunger without lateral support, and may be fastened in place on the anvil by a drop of shellac. The average of the number of blows on the five briquettes, required to produce failure in each case, is the result to be reported, and is the "coefficient of cementation".

Crushing Strength of Rock or Slag.

Cylinders shall be cut from a suitable block of the material to be tested, each of which cylinders shall, as nearly as practicable, be 5 cm. (2 in.) in diameter and 10 cm. (4 in.) in length. After cutting, the dimensions of each cylinder shall be accurately measured and recorded. Each cylinder shall then be subjected to compression, and the ultimate stress at which its failure occurs shall be noted. This stress divided by the average area in cross-section of the cylinder in square inches shall be reported. It is desirable that the test of the material shall be made on at least three such cylinders separately, and the average of the three or more specimens shall be taken as the average resistance to crushing of the material. In making the test, the cylinder shall be fixed in the testing machine so as to be unsupported on its sides and rest squarely on its ends, and the compressive stress shall be applied cumulatively. The ends of the cylinder shall be at right angles to its long axis, and the blocks or pieces of the machine in contact with the ends of the cylinder and through which the pressure is transmitted shall have such position and freedom of movement in the machine as will insure the application of the stress directly along or parallel to the long axis of the cylinder.

Mechanical Analysis of Broken Stone, Broken Slag, or Gravel.*

The method shall consist of, first, drying at not more than 110° cent. (230° Fahr.) to a constant weight a sample weighing in pounds six times the diameter in inches of the largest holes required; second, passing the sample through such of the following sized screens having circular openings as are required or called for by the specification, screens to be used in the order named: 8.89 cm. (3½ in.), 7.62 cm. (3 in.), 6.35 cm. (2½ in.), 5.08 cm. (2 in.), 3.81 cm. (1½ in.), 3.18 cm. (1¼ in.), 2.54 cm. (1 in.), 1.90 cm. (¾ in.), 1.27 cm. (½ in.), and 0.64 cm. (¼ in.); third, determining the percentage by weight retained on each screen; fourth, recording the mechanical analysis in the following manner:

Percentage passing 0.64-cm. (¼-in.) screen, ... =
Percentage passing 1.27-cm. (½-in.) screen and retained on 0.64-cm. (¼-in.) screen................................ =
Percentage passing 1.90-cm. (¾-in.) screen and retained on 1.27-cm. (½-in.) screen................................ =
Percentage passing 2.54-cm. (1-in.) screen and retained on 1.90-cm. (¾-in.) screen................................ =
.... =

100.00

* Adapted from method proposed in 1914 by Committee D-4, "Standard Tests for Road Materials", of the Am. Soc. for Testing Materials.

Mechanical Analysis of Sand or Other Fine Highway Material.

The method shall consist of: First, drying at not more than 110° cent. (230° Fahr.) to a constant weight a sample weighing 100 grammes; second, passing the sample through each of the following mesh sieves, the sieves to be used in the order named:

Meshes per linear inch (2.54 cm.)	Diameter of Wire. Inches.	Diameter of Wire. Millimeters.
10	0.027	0.6858
20	0.0165	0.4191
30	0.01375	0.34925
40	0.01025	0.26035
50	0.009	0.22865
80	0.00575	0.1460
100	0.0045	0.1143
200	0.00235	0.05969

third, determining the percentage by weight retained on each sieve, the sifting being continued on each sieve until less than 1% of the weight retained on each sieve shall pass through the sieve during the last minute of sifting; fourth, recording the mechanical analysis in the following manner:

Percentage passing 200-mesh sieve........................ =
Percentage passing 100-mesh sieve and retained on 200-mesh sieve........................ =
Percentage passing 80-mesh sieve and retained on 100-mesh sieve........................ =
Percentage passing 50-mesh sieve and retained on 80-mesh sieve........................ =
.... =

100.00

Mechanical Analysis of Mixtures of Sand or Other Fine Highway Material with Broken Stone, Broken Slag, or Gravel.

The method shall consist of: First, drying at not more than 110° cent. (230° Fahr.) to a constant weight, a sample weighing in pounds six times the diameter in inches of the largest holes required; second, separating the sample by the use of a 10-mesh sieve (American Society for Testing Materials standard sieve); third, examining the portion retained on the 10-mesh sieve in accordance with the method for making a "Mechanical Analysis of Broken Stone, Broken Slag, or Gravel"; fourth, examining the portion passing the 10-mesh sieve in accordance with the method for making a "Mechanical Analysis of Sand or Other

Fine Highway Material"; fifth, recording the mechanical analysis in the following manner:

Percentage passing 200-mesh sieve........................ =
Percentage passing 100-mesh sieve and retained on 200-mesh sieve.. =
Percentage passing 80-mesh sieve and retained on 100-mesh sieve.. =
.... =
Percentage passing 10-mesh sieve and retained on 20-mesh sieve.. =
Percentage passing 0.64-cm. ($\frac{1}{4}$-in.) screen and retained on 10-mesh sieve.. =
Percentage passing 1.27-cm. ($\frac{1}{2}$-in.) screen and retained on 0.64-cm. ($\frac{1}{4}$-in.) screen................................ =
Percentage passing 1.90-cm. ($\frac{3}{4}$-in.) screen and retained on 1.27-cm. ($\frac{1}{2}$-in.) screen................................ =
.... =

100.00

Voids in Mineral Aggregates.*

"The voids in mineral aggregates shall be determined by the Cone Specific Gravity Method. In the method of making the determination of voids, as hereinafter described, there shall be used a truncated cone made of No. 18, B. & S.-gauge galvanized steel with caulked seams, and having the following dimensions: over-all diameter of bottom, 25.4 cm. (10 in.); over-all height, 25.4 cm. (10 in.); inside diameter of opening, 7.6 cm. (3 in.). The test shall be made in the following manner: First, thoroughly mix the aggregate by rolling on paper; second, fill the cone with aggregates avoiding segregation; third, compact aggregate in cone by oscillation on edge of cone resting on wooden floor, wooden box, or block of wood, and use cotton waste pressed against surface of aggregate to prevent segregation during oscillation; fourth, continue to add aggregate and compact until the cone is full of thoroughly compacted aggregate, which process will require from 300 to 500 oscillations; fifth, weigh cone with aggregate; sixth, weigh cone empty; seventh, weigh cone full of clean water; eighth, determine the specific gravity of aggregate; ninth, the percentage of voids in the aggregate shall be calculated by the following formula:

$$\text{Percentage of voids} = \left(1 - \frac{C - A}{(B - A)\, D}\right) 100$$

in which A = the weight in grams of the cone; B = the weight in grams of the cone filled with water; C = the weight in grams of the cone filled with compacted aggregate; D = the specific gravity of the aggregate."

* Proposed in 1915 by Committee D-4, "Standard Tests for Road Materials", of the Am. Soc. for Testing Materials.

Rattler Test for Paving Brick*

Construction of the Rattler.

General Design.—The machine shall be of good mechanical construction, self-contained, shall conform to the following details of material and dimensions, and shall consist of barrel, frame, and driving mechanisms as herein described.

The Barrel.—The barrel of the machine shall be made up of the heads, headliners, staves, and stave-liners.

The Frame and Driving Mechanism.—The barrel shall be mounted on a cast-iron frame of sufficient strength and rigidity to support it without undue vibration. It shall rest on a rigid foundation with or without the interposition of wooden plates, and shall be fastened thereto by bolts at not less than four points. It shall be driven by gearing having a ratio of driver to driven of not less than one to four.

The Abrasive Charge.—The abrasive charge shall consist of cast-iron spheres of two sizes. When new, the larger spheres shall be 9.52 cm. (3.75 in.) in diameter and shall weigh approximately 3.40 kg. (7.5 lb.) each. Ten spheres of this size shall be used. These shall be weighed separately after each ten tests, and if the weight of any large sphere falls to 3.175 kg. (7 lb.), it shall be discarded and a new one substituted; provided, however, that all of the large spheres shall not be discarded and substituted by new ones at any single time, and that so far as possible the large spheres shall compose a graduated series in various stages of wear. When new, the smaller spheres shall be 4.762 cm. (1.875 in.) in diameter and shall weigh approximately 0.43 kg. (0.95 lb.) each. In general, the number of small spheres in a charge shall not fall below 245 nor exceed 260. The collective weight of the large and small spheres shall be as nearly 136 kg. (300 lb.) as possible. No small sphere shall be retained in use after it has been worn down so that it will pass a circular hole 4.45 cm. (1.75 in.) in diameter, drilled in an iron plate 0.64 cm. ($\frac{1}{4}$ in.) in thickness, or weigh less than 0.34 kg. (0.75 lb.). Further, the small spheres shall be tested, by passing them over the above plate or by weighing, after ten tests, and any which pass through or fall below the specified weight, shall be replaced by new spheres; provided, further, that all the small spheres shall not be rejected and replaced by new ones at any one time, and that, so far as possible, the small spheres shall compose a graduated series in various stages of wear. At any time that any sphere is found to be broken or defective, it shall at once be replaced.

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* Proposed by the National Paving Brick Manufacturers Association and adapted from the "Standard Specifications for Paving Brick", adopted in 1915 by the Am. Soc. for Testing Materials.

The iron composing these spheres shall have a chemical composition within the following limits:

Combined carbon	Not less than	2.50	per cent.
Graphitic carbon	Not more than	0.25	"
Silicon	" " "	1.00	"
Manganese	" " "	0.50	"
Phosphorus	" " "	0.25	"
Sulphur	" " "	0.08	"

Operation of the Test.

The Brick Charge.—The number of bricks per test shall be ten for all bricks of so-called "block-size", having dimensions which fall between 20.32 and 22.86 cm. (8 and 9 in.) in length, 7.62 and 9.52 cm. (3 and $3\frac{3}{4}$ in.) in breadth, and 9.52 and 10.8 cm. ($3\frac{3}{4}$ and $4\frac{1}{4}$ in.) in thickness. No brick should be selected as part of a regular test that would be rejected by any other requirements of the specifications under which the purchase is made. (*Note by Committee.*—Each brick should be marked by small holes drilled in one of the faces of the brick, and the initial weight of each brick composing the charge should be determined.)

Speed and Duration of Revolution.—The rattler shall be rotated at a uniform rate of not less than 29.5 nor more than 30.5 rev. per min., and 1 800 revolutions shall constitute the test. A counting machine shall be attached to the rattler for recording the revolutions. A margin of not more than 10 revolutions will be allowed for stopping. Only one start and stop per test is generally acceptable. If, from accidental causes, the rattler is stopped and started more than once during a test, and the loss exceeds the maximum permissible under the specifications, the test shall be discarded and another made.

The Scales.—The scales must have a capacity of not less than 136 kg. (300 lb.), must be sensitive to 14.17 grammes (0.5 oz.), and must be tested by a standard test weight at intervals of not less than every ten tests.

The Results.—The loss shall be calculated in percentage of the initial weight of the brick composing the charge. In weighing the rattled brick, any piece weighing less than 0.45 kg. (1 lb.) shall be rejected. (*Note by Committee.*—The loss for each brick should also be calculated in percentage of the initial weight of each brick composing the charge.)

Absorption of Water by Wood Blocks After Treatment.

Five blocks of average character shall be heated in an oven to a temperature of 110° cent. (230° Fahr.) for 3 hours, then weighed, and immersed in water for the same length of time. At the end of this

time, they shall be taken out, wiped dry, and weighed, the difference in weight before and after immersion, calculated on the weight after heating, being the percentage of absorption.

Specific Gravity at 38° Cent. of Wood Block Preservative.*

A standardized hydrometer shall be used. A set of two with ranges 1.00 to 1.08, and 1.07 to 1.15 will suffice. Before taking the specific gravity, the oil in the cylinder should be stirred thoroughly with a glass rod, and this rod when withdrawn from the liquid should show no solid particles at the instant of withdrawal. Care should be taken that the hydrometer does not touch the sides or bottom of the cylinder when the reading is taken, and that the oil surface is free from froth and bubbles. If the specific gravity is determined at a higher temperature than desired, correction should be made by adding 0.0008 to the reading for each degree centigrade excess of temperature.

Solubility in Benzol or Chloroform of Wood Block Preservative.

From 5 to 10 grammes of the water-free oil is weighed out into a weighed 100-c.c. (3.38-oz.) beaker. 50 c.c. (1.69 oz.) of the solvent is added, and the solution is passed through a weighted 9-cm., C. S. & S., No. 575 filter paper in a short-stemmed funnel, the filtrate being passed into the flask to be subsequently used for the hot extraction. The beaker is washed clean from all soluble matter, dried, and weighed. The funnel, with filter paper and contents, is then placed in a N. Y. T. L. or Underwriter's form of glass extraction apparatus, and heat is applied from a water-bath or hot plate until the extraction is complete and the filtrate runs through colorless. The filter and contents is then dried and weighed. The increase in weight is added to the increase in weight of the beaker, if any, the result being the weight of the insoluble matter. The weight of the insoluble matter thus found subtracted from the weight of the material taken for analysis. The difference in weight is the weight of the soluble matter, from which the percentagè is calculated.

Water Content of Wood Block Preservative.

From 250 to 300 c.c. (8.45 to 10.14 oz.) of the oil is weighed out into a 500-c.c. glass retort, or into a small copper still, provided with a distilling head. Heat is applied with a ring burner, starting with a small flame at the top of the still, and gradually lowering it until all the water has been driven off. The distillate of oil and water is collected in a graduated separatory funnel, the volume of water, in cubic centimeters, is read, and its percentage figured by volume. The water is then drawn off and the oils are returned to the residue in the

* Modification of method proposed in 1915 by Committee D-7 on "Standard Specifications for Timber", of the Am. Soc. for Testing Materials.

still. The contents of the still shall have cooled to below 100° cent. (212° Fahr.) before the oils are returned, and they shall be well stirred and mixed with the residue.

Distillation Test for Wood Block Preservative.*

Apparatus for Distillation Test.

Retort.—This shall be a tabulated Jena glass retort of the usual form, with a capacity of 250 to 290 c.c. (8.45 to 9.8 oz.). The capacity shall be measured by placing the retort with the bottom of the bulb and the end of the offtake in the same horizontal plane, and pouring water into the bulb through the tubulature until it overflows the offtake. The quantity remaining in the bulb shall be considered its capacity.

Shield.—An asbestos shield shall be used to protect the retort from air currents and to prevent radiation. This may be covered with galvanized iron, as such an arrangement is more convenient and more permanent.

Receivers.—Erlenmeyer flasks of from 50 to 100 c.c. capacity are of the most convenient form.

Thermometer.—The thermometer shall be of glass, well annealed, and shall undergo no serious change at the zero point when heated up to 400° cent. The space above the mercury column shall be filled with gas, either carbon dioxide or nitrogen, and the thermometer shall have an expansion chamber at the top. The scale shall read from 0 to 400° cent., in graduations of 1° cent., which shall be etched on the stem. The tip of the thermometer shall carry a ring for the purpose of attaching tags. The thermometer shall have the following dimensions:

Total length, 375 mm.; tolerance, 10 mm.
Bulb length, 14 mm.; tolerance, 1 mm.
Distance from zero mark to bottom of bulb, 30 mm.; tolerance, 4 mm.
Scale length from zero mark to 400° cent., 295 mm.; tolerance, 5 mm.
Diameter of stem, 7 mm.; tolerance, 1 mm.
Diameter of bulb, 6 mm.; tolerance, 1 mm.

When standardized, the accuracy of such standardization should be as follows:

Up to 200° cent.	. . . . to the nearest	0.5° cent.
200 to 300° "	. . . . " " "	1.0° "
300 to 360° "	. . . . " " "	1.5° "

Assembling for Distillation Test.

The retort shall be supported on a tripod or rings over two sheets of 20-mesh gauge, 15.24 cm. (6 in.) square. It shall be connected to

* Modification of method proposed in 1915 by Committee D-7 on "Standard Specifications for Timber" of the Am. Soc. for Testing Materials.

the condenser tube by a tight cork joint. The thermometer shall be inserted through a cork in the tubulature, with the bottom of the bulb 1.27 cm. ($\frac{1}{2}$ in.) from the surface of the oil in the retort. The exact location of the thermometer bulb shall be determined by placing a vertical rule graduated in divisions not exceeding 0.16 cm. ($\frac{1}{16}$ in.) back of the retort when the latter is in position for the test, and sighting the level of the liquid and the point for the bottom of the thermometer bulb. The distance from the bulb of the thermometer to the outlet end of the condenser tube shall be not more than 60.96 cm. (24 in.) nor less than 50.8 cm. (20 in.). The burner should be protected from drafts by a suitable shield or chimney.

Distillation Test.

Exactly 100 grammes of oil shall be weighed into the retort, the apparatus shall be assembled, and heat applied. The distillation shall be conducted at the rate of at least one drop, and not more than two drops, per second, and the distillate collected in weighed receivers. The condenser tube shall be warmed whenever necessary to prevent accumulation of solid distillates. Fractions shall be collected at the following points:

Up to 170° cent. (338° Fahr.), 170-200° cent. (338-392° Fahr.), 200-210° cent. (392-410° Fahr.), 210-235° cent. (410-455° Fahr.), 235-270° cent. (455-518° Fahr.), 270-300° cent. (518-572° Fahr.), 300-315° cent. (572-599° Fahr.), 315-355° cent. (599-671° Fahr.).

The receivers shall be changed as the mercury passes the dividing temperature for each fraction. The last receiver shall be removed at 355° cent. (671° Fahr.), and drainage from the condenser, etc., shall not be considered as part of the fraction. For weighing the receivers and fractions, a balance accurate to at least 0.05 gramme shall be used. During the progress of the distillation the thermometer shall remain in its original position. No correction shall be made for the emergent stem of the thermometer.

When any measurable quantity of water is present in the distillate, it shall be separated as nearly as possible and reported separately, all results being calculated on a basis of dry oil. When more than 2% of water is present, water-free oil shall be obtained by separately distilling a larger quantity, returning any oil carried over with the water, and using dried oil for the final distillation. A copper tar still is a convenient implement for obtaining water-free oil.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

PROGRESS REPORT OF THE SPECIAL COMMITTEE ON A NATIONAL WATER LAW*

Authority.—In accordance with a resolution of the Board of Direction adopted at the meeting of June 4th, 1913, a Special Committee of nine members was appointed to investigate and report upon the substance of the following resolution submitted on May 7th, 1913, by Mr. John H. Lewis.

"*Moved:* That the Board of Direction of the American Society of Civil Engineers be and is hereby authorized and directed to appoint a special committee to investigate the advisability of drafting a National Water Law applicable to all navigable, interstate and other waters within the jurisdiction of the United States, and embracing all uses of water, and that such committee be directed to prepare a preliminary draft of such law for submission at some regular meeting of the Society, if, in their judgment, it appears advisable."

The Special Committee thus created has the honor to submit the following report:

Inadequacy of Existing Laws.—Such national law as exists in the United States relative to the control and use of water has been in general the product of conditions now largely obsolete, and is neither consistent with nor adequate to the necessities of the present. For example, if any use of water has been given precedence by our National Statutes and Courts, it is that for navigation; but, excluding the Great Lakes and their connecting waters, and the Hudson, Mississippi, and Ohio Rivers, so far as the fresh waters of the country are concerned, of all uses to which they can be put, navigation is now the lowest in importance.

The reason and purpose of all law and government being the attainment of the greatest good to the greatest number of those governed, therefore as conditions change, laws and government must change, else they no longer conduce to the contemplated end.

* To be presented to the Annual Meeting, January 19th, 1916.

Relative Importance of Uses.—Those uses of water which vitally affect the life and health of humanity have precedence over all other uses, through the operation of the inexorable laws of Nature. Next come those uses necessary to the production of food supplies; and after these, in order of importance, follow the application of water to the essentials and conveniences of civilization.

(*a*) The first use of water in importance and the one truly paramount, namely, that for the domestic or household requirements of man—one without which life itself must disappear—was not recognized as of sufficient consequence for consideration when the rights of navigation received their present prominence in this country. At the time of the adoption of the Federal Constitution, outside of New York City and Bethlehem, Pa., municipal or public water supplies appear to have been non-existent in America.

(*b*) The second use of water, in present importance, is that for the watering of live stock and the production of crops, upon which two factors depend the food supplies of the nation. The former of these applications is long recognized and well established, but the latter, now reaching its highest development in the vast irrigation systems of the arid and semi-arid States, is a use almost unknown in this country prior to the Civil War. Co-ordinate with this use, and as a necessary adjunct in many cases, is the drainage of lands rendered unproductive by an excess of water.

(*c*) The third in relative importance is the use of water in the disposal of city sewage, for, whether with or without preliminary treatment, the sewage from inland cities must of necessity be carried away and finally disposed of in natural waterways, which are a part of the great circulatory system of the country; and the right to the use of water for this purpose must be clearly recognized.

(*d*) The fourth use is that for manufacturing, and for the generation of power to be used in the production of the requisites of civilization. The former is practically inseparable from the domestic uses in an ordinary municipality, but the latter is a larger, more specific and more easily distinguished use and is commonly designated power development. Closely related to the above is the preservation of property by protection against floods and the regulation of the stream flow to this end, as well as with a view to an increased utilization of the water.

(*e*) The fifth use is that for the transportation of the products of agriculture and the materials of manufacture, and for the convenience of travel, designated as navigation.

Of the uses above noted, the first three are essentially uses at large, in that their direct benefits are shared by the community in general and with the least intervention of profit-making intermediaries. The

last two are essentially commercial uses, developed chiefly for individual gain, and should be subject to the requirements of the former. All but the first should yield to the rights of eminent domain exercised in favor of those uses higher in rank.

General Principles.—Growing out of these fundamental conceptions are certain principles which when clearly stated should be generally approved as forming the foundations for a water law or body of laws. Of such principles the following are enumerated:

Public Welfare.—The public welfare as to any particular project on any interstate or navigable water is paramount.

Unity.—Each river system, from its head-waters to its mouth, is a natural unit, regardless of political boundaries.

Plan.—It is desirable that there be a comprehensive plan for the largest immediate use consistent with the best future development of each river system, the plan being such that it can ultimately be carried to completion with the accomplishment of the greatest public good.

Construction.—Such works as may materially modify the conditions of a river system should be carried into effect in a manner consistent with the largest public welfare.

Interstate Rights.—On interstate waters the principle "first in time is first in right" should be recognized as applying to rights already acquired, irrespective of political boundaries, and rights to the use of unappropriated water should be equitably apportioned between the States on the basis of the highest economic use.

Economic Use.—The expression "highest economic use of water" may be defined to mean that use by which the product of the utilization of the water shall add most to the wealth or welfare, or both, of the community at large, taking into account both gains and losses, direct and indirect, and irrespective of whether the utilization be accomplished through private enterprise or directly by the commonwealth.

The above described general principles have been agreed upon by the Special Committee as fundamental to a National Water Law.

The Committee finds that the powers of Congress are so restricted by the lack of Constitutional authority that the enactment of a comprehensive law is now impossible, and the end in view in the appointment of this Committee can be achieved only as the result of an extensive campaign of education. Believing that the Engineer (who throughout history, though often without due recognition, has stood at the forefront of human progress) is best qualified and most interested to point the way to a rational administration of this greatest of our natural

resources, the Committee urges upon the membership of the Profession a careful study of the principles and policies herein set forth, with a view to their promulgation to those less qualified to judge of the merits of the great questions involved.

The investigations of this Committee have brought into clear relief the fact that the water resources of the country now demand for their development that definite provision be made for uses and conditions which have hitherto received but scant attention. They have also clearly demonstrated that the best utilization of these resources requires the general adoption of constructive policies rather than those of a restrictive character, which have so far prevailed, and which have resulted in the present general stagnation of development in hydraulic lines.

Among the present requirements, the most urgent are provision for water storage, for the use of waters in the disposal of sewage, for the drainage of lands, for the ascertaining and recording of rights, and for related administrative matters.

The full utilization of the waters of the country will involve, in many cases, the provision of storage for equalizing flow to reduce flood damage, and benefit power production and navigation, and in frequent instances may call for diversion of water from one drainage basin to another. In determining the economic utility of any proposed plan, all losses and benefits which may accrue must be considered, and provision should be made for the exercise of the right of eminent domain in one State for the benefit of citizens of other States, in order that improvements to the economic conditions of streams may be carried to completion. In all cases the cost of benefits accruing should be assessed to those interests benefited, be they public or private, and all artificial obstacles to water utilization resulting from technical or legal objections, so far as may be practicable, should be removed by legislative action.

The use of interstate and international, as well as of intrastate waters, for the transportation and disposal of city sewage and industrial wastes, stands in such vital relation to the general welfare of the entire country as to justify and demand specific authorization for such use under suitable regulation and control.

The disposal of harmful excess of water from wet lands bears much the same relation to crop production and health in rural communities as the disposal of sewage bears to health and industry in cities; and the right to the use of natural waterways in land drainage should be clearly recognized and firmly established.

As far as possible, and with full consideration of unity of principles and practice, the administration of water laws and regulations should be placed in the hands of local authorities. The right to the

use of waters appropriated for specific purposes should not be transferred or applied to other purposes to the detriment of existing rights, beneficial use being the basis, the measure, and the limit of the right to the use of water.

Public record of existing rights should be provided for as a basis for the proper administration of the waters of the country, and in cases where important streams flow from one State into another there should be an interchange of such records between the States.

Much of the legislation required for the best utilization of the water resources of the country must of necessity originate in the States rather than in the General Government, though a part is undoubtedly within the scope of Congressional action; but any code of law upon this subject, to be consistent and harmonious and really effective, must recognize and embody the principles herein developed.

Conclusion.—The drafting of a National Water Law seems to your Committee inadvisable until the members of this Society and engineers in general have had opportunity to consider and discuss the principles herein set forth.

It is deemed important to consult committees of other professional organizations interested in water legislation, with a view to securing wider discussion and the diffusion of ideals, and in order that co-operative action may be had toward the desired results.

To this end the Special Committee recommends that it be continued with instructions to include in its consideration State as well as Federal water laws, and further that it be authorized to enter into co-operative relations with similar committees from other professional organizations.

All of which is respectfully submitted.

F. H. Newell, *Chairman.*
George G. Anderson.
Charles W. Comstock.
Clemens Herschel.
W. C. Hoad.
Robert E. Horton.
John H. Lewis.
Gardner S. Williams.

(Note.—This report was received without the signature of President Marx, and, although he may be willing to sign it, he has not authorized the Secretary to do so for him.—*Secretary.*)

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

PROGRESS REPORT OF THE SPECIAL COMMITTEE ON STEEL COLUMNS AND STRUTS*

To the American Society of Civil Engineers:

The Special Committee, authorized by vote of this Society "to consider and report upon the design, ultimate strength, and safe working values of steel columns and struts", presents the following report of progress.

During the past year, your Committee has held three regular meetings, and the minutes of these meetings have been reported to the Board of Direction for publication in the *Proceedings.*

Since your Committee made its last report, the Sub-Committee on Built-up Columns, of the Committee on Iron and Steel Structures, of the American Railway Engineering Association, has presented its progress report, giving the results of tests on its series of columns. The American Railway Engineering Association Committee and your Committee are co-operating in their test programmes with the Bureau of Standards, S. W. Stratton, Director, and the report published in the American Railway Engineering Association *Proceedings,* Vol. 16, 1915, pages 636-654, should be studied along with our programme.

The last report of your Committee showed that some 70 columns had been tested. During the past year, the Bureau of Standards has tested 110 additional columns, making a total of 180, which are now available for study and discussion. These include the light and heavy sections shown in our last report,† 18 Bethlehem H-columns, light and heavy sections, and 18 Carnegie mine section, H-columns, light sections. The Bethlehem H-columns and their properties, also the 18 Carnegie, mine section, H-columns, 9 columns which are 6-in., 23.8 lb. per ft., and

* To be presented to the Annual Meeting, January 19th, 1916.

† *Proceedings,* Am. Soc. C. E., for December, 1914, pp. 3057 and 3059.

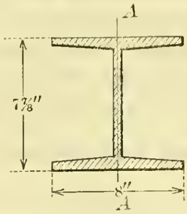
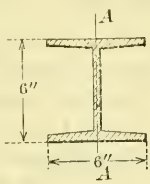
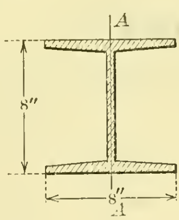
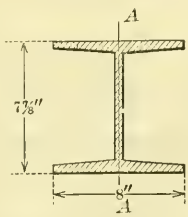
9 columns which are 8-in., 34.0 lb. per ft., are shown on Table 1. These mine sections were donated to the Bureau of Standards for test purpose by the Carnegie Steel Company.

Your Committee has carefully followed the testing of the columns with the Bureau of Standards, so that our joint council might be able to suggest improvements in the methods of testing, or changes in the programme. Our studies of the records of the 70 tests reported last year showed that we would be assisted in interpreting the results if comparisons could be made with columns of different slenderness ratios than the 50, 85, and 120 $\frac{l}{r}$ provided for in the original programme. The original selection of these slenderness ratios came about because 50 $\frac{l}{r}$ represented the average short column, and 120 $\frac{l}{r}$ the extreme of long columns in common practice, 85 $\frac{l}{r}$ being half way between the above extremes. It was felt that the difference in slenderness ratio of 35 $\frac{l}{r}$ was small enough to give ample information of the character of columns anywhere within this range. It was therefore decided to extend the series, maintaining the same progressive ratio of 35 $\frac{l}{r}$, and adopting a new series with a slenderness ratio of 155 $\frac{l}{r}$. Your Committee decided that it was not necessary to carry out this series with the 9 types of columns in the original programme, and felt that 3 types, Nos. 1, 2, and 4, would give sufficient information from which to draw conclusions. The sections of these columns and their properties are shown in Table 2.

When your Committee published the results of column tests in a Progress Report,* it felt that the tests of short struts with a slenderness ratio of less than 50 $\frac{l}{r}$, as recorded in this Progress Report, provided sufficient information to write the portion of a column formula within this range. After the first series of tests on light columns was completed it was suggested that it would be possible to take some of the long columns and cut pieces from them which would be available for tests of short struts. Three light and three heavy struts were cut from the plate and angle section column, Type 1, so as to give a slenderness ratio of 20 $\frac{l}{r}$. These were tested and the results are shown on Table 4 as tests 84-*A*, 84-*B*, 85-*A*, and 98-*A*, 98-*B*, 99-*A*.

* *Transactions*, Am. Soc. C. E., Vol. LXVI, 1910, p. 401.

TABLE 1.

	Column and section.		Area □″	r	$\frac{l}{r}$	Length. Ft.	Length. Ins.
Light Section.	BETHLEHEM. 8-in H-Column. 8-in. by 31.5 lb.		9.17	1.98	120	19	$9\frac{5}{8}$
			9.17	1.98	85	14	$0\frac{5}{16}$
			9.17	1.98	50	8	3
Light Section.	CARNEGIE. 6-in. H-Column Section H 3. 6 in. by 23.8 lb.		7.0	1.45	120	14	6.03
			7.0	1.45	120	14	6.17
			7.0	1.45	120	14	6.03
			7.0	1.45	85	10	3.22
			7.0	1.45	85	10	3.16
			7.0	1.45	85	10	3.25
			7.0	1 45	50	6	0.45
			7.0	1.45	50	6	0.47
			7.0	1.45	50	6	0.48
Light Section.	CARNEGIE. 8-in. H-Column Section H 4 8 in. by 34 lb.		10.0	1.87	120	18	8.52
			10.0	1.87	120	18	8.51
			10.0	1.87	120	18	8.51
			10.0	1.87	85	13	2.98
			10.0	1.87	85	13	3.00
			10.0	1.87	85	13	2.97
			10.0	1.87	50	7	9.48
			10.0	1.87	50	7	9.39
			10.0	1.87	50	7	9.38
Heavy Section.	BETHLEHEM. 8-in. H-Column 8 in. by 62 lb.		18.27	2.09	120	20	$10\frac{13}{16}$
			18.27	2.09	85	14	$9\frac{5}{8}$
			18.27	2.09	50	8	$8\frac{1}{2}$

It will be seen that the system of numbering indicates the numbers of the original test columns from which these pieces were cut.

Later in this report, the results of the tests are arranged to show the variation in unit ultimate compression between light and heavy

sections. It was thought advisable to begin an investigation to learn whether the drop in ultimate strength of the heavy sections was progressive as the thickness of the metal increased, and additional columns, with very much heavier metal are to be provided by the Bureau of Standards. These columns, with their areas and properties, are given in Table 3. It will be noted that the Committee has chosen one set of three columns of Type 1, which is a plate and angle I-shape column, with metal $\frac{13}{16}$ and $\frac{7}{8}$ in. thick, and an area of 28.61 sq. in., as against 11.475 sq. in. for the original light sections, and 22.19 sq. in. for the original heavy sections. Only one slenderness ratio was chosen, and this was made 85 $\frac{l}{r}$, which is the average of the three ratios originally provided. Three slenderness ratios, 50, 85, and 120 $\frac{l}{r}$, have also been chosen for the columns of Type 5, an 8-in. Bethlehem **H**-section, with an area of 26.64 sq. in., as against 9.17 sq. in. for the original light section, and 18.27 sq. in. for the original heavy section. For test purposes, this section is free from the complications which might be produced by the riveting. When the tests are completed, this additional series will be of value when compared with the lighter sections of the same types.

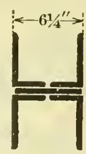
TABLE 2.

Section.	Weight per ft-lb.	Area, sq. in.	r	$\frac{l}{r}$	Length.	Sketch.
4 Ls 5 × 3 × 5/16 1 Pl. 6 × 5/16	39.18	11.475	2.24	155	28'-11 13/16"	
4 Ls 5 × 3 × 5/8 1 Pl. 6 × 5/8	75.55	22.19	2.36	155	30'-5 13/16"	
2 [s 6" × 10½* 2 Pls. 8 × ¼	34.60	10.18	2.31	155	29'-10 1/16"	
2 [s 6" × 15½* 2 Pls. 8 × ½	58.20	17.12	2.33	155	30'-1⅛"	
2 [s 8" × 11¼* 1 I 8" × 18*	40.50	12.03	2.38	155	30'-8⅞"	
2 [s 8" × 18¾* 1 I 8" × 20½*	58.00	17.05	2.32	155	29'-11⅝"	

The Progress Report presented in 1915 gave complete records of the tests of two typical columns, Nos. 37 and 54. These records are republished in this report as Plates LXXIII and LXXIV. The Bureau of Standards has carried out all the tests with the same attention to minute detailed measurements, and your Committee has had the benefit of these complete data. The voluminousness of these records precludes their complete publication by the Society, but arrangements will ultimately be made whereby copies will be filed in the Library of the Society, to be available for those who desire to study them in detail.

In Tables 4 to 8 your Committee shows an abstract of the results of the tests to date. It will be noted that these tables are arranged so that the light and heavy sections of each type are placed together, so that they may be readily compared.

TABLE 3.

Section.	Weight per ft-lb.	Area, sq. in.	r	$\frac{l}{r}$	Length.	Sketch.
4 ∟s 5 × 3 × 13/16 1 Pl. 6 × 7/8	97.45	28.61	2.29	85	16′-2⅝″	
8″ Beth.........	90.50	26.64	2.17	50	9′-0½″	
8″ Beth.........	90.50	26.64	2.17	85	15′-4 7/16″	
8″ Beth.........	90.50	26.64	2.17	120	21′-8⅜″	

In the original programme, your Committee desired to eliminate as much as possible the variables which have thus far served to complicate former test programmes. The study of these former programmes showed such marked variation in the character of the steel used, the choice of shapes, the slenderness ratios, and the methods of testing, that your Committee despaired of being able to carry out the original instructions of determining ultimate strengths and safe working loads, with the information at hand. At that time, the Bureau of Standards was completing the new 2 300 000-lb. testing machine, and offered to provide the material for a comprehensive programme of tests. Your Committee originally planned ten different shapes of columns, covering commercial forms, with the idea of varying these

TABLE 4.

ABSTRACT OF RECORD OF COLUMN TESTS

TYPES I & IA — I & IA SPECIAL

Type	Test No.	Section	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit ultimate Strength.	Average ultimate Strength.	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
	8	Light Sections	11.13	2.24	9'-4"	50	32400		0.035 S.	0.000
	38	A	11.17	"	"	"	32800	32700	0.07 S.	0.1000 Up.
	77		11.13	"	"	"	33000		0.050 N.	0.0313 Up
	18		11.49	"	15'-10 7/16"	85	32100		0.12 S	0.05 Up.
I.	84	A	11.49	"	"	"	31400	31200	0.0252 S.	0.1281 Down.
	85	4L^s 5"x3"x5/16" = 9.60□"	11.35	"	"	"	30000		0.0626 N.	0.1368 Up.
		1Pl. 6"x5/16" = 1.875								
	30	Total = 11.475□"	11.20	"	22'-4 13/16"	120	28600		0.000	0.060 Down.
	56		11.14	"			27600	28300	0.1175 S.	0.1091 Up.
	69		11.30	"			28700		0.0135 N.	0.1785 Down.
	93	Heavy Sections	22.25	2.36	9'-10"	50	29000		0.0080 S.	0.0574 Down.
	96		22.13	"	"	"	28900	29200	0.0731 N.	0.0457 Up.
	97		22.28	"	"	"	29700		0.0653 N.	0.0002 Up.
		A — A								
	91		22.134	"	16'-8 5/8"	85	28000		0.0475 S.	0.0883 Up.
IA.	94		21.92	"	"	"	28200	28100	0.4053 N.	0.2636 Up.
	95	4L^s 5"x3"x5/8" = 18.44□"	22.05	"	"	"	28000		0.0764 N.	0.0825 Down.
		1Pl. 6"x5/8" = 3.75								
	92	Total = 22.19□"	22.15	"	23'-7 3/16"	120	25000		0.1430 S.	0.2923 Down.
	98		22.10	"	"	"	25500	25400	0.3863 N.	0.0762 Up.
	99		22.10	"	"	"	25800		0.1326 N.	0.0606 Down.
	85A	Light Sections	11.35	2.24	3'-8 3/4"	20	35100		0.0089 N.	0.0600 Down.
	84A		11.49	"	"	"	38000	36400	0.0230 S.	0.0313 Up.
	84B		11.49	"	"	"	36100		0.0141 S.	0.0120 Down.
I Spec.		as above								
	99A	Heavy Sections	22.10	2.36	3'-11 3/16"	20	45000		0.0209 S.	0.0157 Down.
	98A		"	"	"	"	44100	44400	0.0177 N.	0.0104 Down.
	98B		"	"	"	"	44100		0.0031 N.	0.0229 Up.
IA Spec.		as above								

TABLE 5.

ABSTRACT OF RECORD OF COLUMN TESTS
TYPES 2 & 2A – 3 & 3A

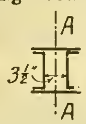
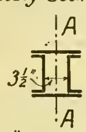
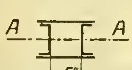
Type	Test no.	Section	Actual Area	Radius of Gyration	Length.	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
	11	Light Sections.	10.06	2.31	9'-7½"	50	33100		0.08 S.	0.00
	41		10.15	"	"	"	34000	33900	0.0024 N.	0.0374 Up.
	43		10.06	"	"	"	34500		0.0460 S.	0.0441 Up.
	21		9.92	"	16'-4⅜"	85	33700		0.12 S.	0.04 Up.
2	59		9.87	"	"	"	32300	32600	0.1195 N.	0.0195 Up.
	65	2-6" [s x 10.5-lb. = 6.18 □"	10.05	"	"	"	31700		0.0627 N.	0.0927 Down
		2 Pl.s 8" x ¼" = 4.00								
	32	Total = 10.18 □"	10.01	"	23'-1 3/16"	120	28300		0.27 S.	0.12 Up.
	54		10.09	"	"	"	28700	29300	0.2750 S.	0.0583 Up.
	70		10.07	"	"	"	30900		0.1725 N.	0.0144 Up.
	101	Heavy Sections.	17.07	2.33	9'-8½"	50	32200		0.0318 N.	0.0360 Up.
	124		17.07	"	"	"	32500	32300	0.0052 N.	0.0044 Up.
	165		16.96	"	"	"	32300		0.1905 S.	0.0131 Up.
	145		17.07	"	16'-6 1/16"	85	30700		0.0416 N.	0.0820 Down.
2A	146		17.02	"	"	"	30000	30600	0.0606 S.	0.0731 Up.
	151	2-6" [s x 15.5-lb. = 9.12 □"	17.11	"	"	"	31000		0.0312 S.	0.0078 Down
		2 Pl.s 8" x ½" = 8.00								
	135	Total = 17.12 □"	17.04	"	23'-3⅝"	120	28500		0.0668 S.	0.0298 Up.
	136		17.02	"	"	"	28800	28100	0.1315 S.	0.1942 Up.
	137		17.08	"	"	"	27000		0.1503 N.	0.0339 Down.
	1	Light Sections.	8.71	2.34	9'-9"	50	33800		0.01 N.	0.06 Up.
	2		8.80	"	"	"	34300	34100	0.01 S.	0.04 Up.
	42		8.71	"	"	"	34300		0.0418 N.	0.0306 Up.
	14		8.71	"	16'-6⅞"	85	32500		0.10 N.	0.06 Down.
3	87		8.69	"	"	"	31900	32400	0.0764 S.	0.1994 Down.
	88	2-5" [s x 6.5-lb. = 3.90 □"	8.60	"	"	"	32900		0.0470 S.	0.1774 Up.
		2 Pl.s 9½" x ¼" = 4.75								
	24	Total = 8.65 □"	8.79	"	23'-4 13/16"	120	31600		0.13 N.	0.01 Up.
	35		8.70	"	"	"	30200	30600	0.17 S.	0.08 Down.
	47		8.74	"	"	"	30000		0.2056 N.	0.1404 Down.
	105	Heavy Sections.	16.44	2.39	9'-11½"	50	29800		0.0084 N.	0.1044 Down.
	168		16.45	"	"	"	29300	29500	0.0077 N.	0.0170 Up.
	171		16.39	"	"	"	29500		0.0126 N.	0.0228 Up.
	147		16.41	"	16'-11⅛"	85	28000		0.0574 S.	0.0721 Down.
3A	148		16.39	"	"	"	28000	28000	0.0313 N.	0.0470 Down.
	150	2-5" [s x 11.5-lb. = 6.76 □"	16.37	"	"	"	28000		0.1159 N.	0.0835 Up.
		2 Pl.s 9½" x ½" = 9.50								
	138	Total = 16.26 □"	16.50	"	23'-10 13/16"	120	28000		0.1848 S.	0.0449 Down.
	139		16.45	"	"	"	26100	26900	0.0919 N.	0.3284 Down.
	143		16.42	"	"	"	26600		0.1201 N.	0.3920 Up.

TABLE 6.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES 4 & 4A – 5 & 5A.

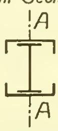
Type	Test no.	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength	Average Ultimate Strength	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
4	6	Light Sections.	11.69	2.38	9′-11″	50	35700		0.11 N.	0.015 Down.
	36		"	"	"	"	36500	36900	0.02 S.	0.04 Up.
	75		"	"	"	"	38600		0.0480 S.	0.0021 Up.
	13		11.66	"	16′-10 5/16″	85	34000		0.04	0.06 Up.
	79		11.73	"	"	"	33900	34000	0.1409 N.	0.0345 Up.
	80	2-8″[s x 11.25-lb = 6.70□″	11.64	"	"	"	34000		0.0626 S.	0.1304 Down.
		1-8″I x 18.00-lb = 5.33								
	26	Total = 12.03□″	11.69	"	23′-9 5/8″	120	32000		0.15 S.	0.10
	45		11.61	"	"	"	30700	31900	0.409 N.	0.0216
	49		11.68	"	"	"	33100		0.0712 N.	0.0094 Up.
4A	102	Heavy Sections.	16.98	2.32	9′-8″	50	30300		0.0158 N.	0.0169 Down.
	166		16.84	"	"	"	29000	29100	0.0504 S.	0.0150 Down.
	169		16.81	"	"	"	28000		0.0311 S.	0.0019 Up.
	152		16.86	"	16′-5 3/16″	85	26000		0.1044 N.	0.0093 Up.
	153		16.89	"	"	"	26400	26600	0.2453 S.	0.0261 Down.
	154	2-8″[s x 18.75-lb = 11.02□″	17.00	"	"	"	27500		0.0835 N.	0.0078 Down.
		1-8″I x 20.50-lb = 6.03								
	132	Total = 17.05□″	16.98	"	23′-2 7/16″	120	23500		0.4176 S.	0.0052 Down.
	133		17.07	"	"	"	23900	23900	0.2349 S.	0.0068 Up.
	134		16.96	"	"	"	24200		0.5283 N.	0.0616 Down.
5	107	Light Sections.	9.62	1.98	8′-3″	50	38000		0.0280 N.	0.0115 Down.
	122		9.65	"	"	"	38000	38000	0.0836 N.	0.0287 Down.
	123		9.65	"	"	"	38000		0.0052 S.	0.0209 Down.
	114		8.61	"	14′-0 5/16″	85	36000		0.1510 S.	0.0078 Down.
	116		9.66	"	"	"	34000	34300	0.1148 S.	0.0085 Up.
	119	Beth. 8″ H Col x 32-lb.	9.59	"	"	"	33000		0.1201 S.	0.0376 Up.
		= 9.17□″								
	110		8.64	"	19′-9 5/8″	120	33900		0.1148 N.	0.0084 Up.
	111		8.68	"	"	"	31000	32000	0.2140 N.	0.0136 Down.
	108		8.61	"	"	"	31000		0.1806 S.	0.0110 Up.
5A	106	Heavy Sections.	17.86	2.09	8′-8 1/2″	50	34000		0.1201 N.	0.0084 Up.
	120		17.82	"	"	"	36100	35400	0.0606 N.	0.0016 Up.
	121		17.86	"	"	"	36000		0.0284 N.	0.0033 Up.
	115		18.05	"	14′-9 5/8″	85	33900		0.0521 N.	0.0151 Down.
	117		17.73	"	"	"	32000	32300	0.0929 N.	0.0054 Down.
	118	Beth. 8″ H Col x 62-lb	17.77	"	"	"	31000		0.0470 S.	0.0277 Up.
		= 18.27□″								
	109		17.76	"	20′-10 13/16″	120	30000		0.1498 N.	0.0037 Up.
	112		17.97	"	"	"	30000	30000	0.2401 S.	0.0219 Down.
	113		17.73	"	"	"	30000		0.1610 N.	0.0221 Down.

TABLE 7.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES 6 & 6A–10 & 10A.

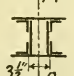
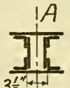
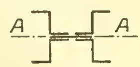
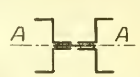
Type	Test no.	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
	10	Light Sections.	13.72	2.31	9'-7½"	50	31200		0.11 N.	0.01 Up.
	40	A	13.78	"	"	"	31500	31600	0.0837 S.	0.0065 Up.
	44		13.72	"	"	"	32200		0.0250 N.	0.0020 Up.
										
	22		13.65	"	16'-4⅜"	85	27800		0.30 N.	0.02 Up.
6	63	A	13.68	"	"	"	29500	29100	0.3040 N.	0.0097 Down.
	64	1-I 10"×25-lb.=7.37□"	13.60	"	"	"	30100		0.1306 S.	0.0166 Down.
		2 Pls. 11"×$\frac{5}{16}$"=6.88								
	31	Total=14.25□"	13.73	"	23'-1$\frac{3}{16}$"	120	26700		0.50 S.	0.04 Up.
	52		13.75	"	"	"	27700	27200	0.1980 S.	0.0190 Down.
	71		13.65	"	"	"	27300		0.1369 S.	0.0013 Up.
	103	Heavy Sections.	23.98	2.47	10'-3½"	50	31200		0.0175 S.	0.0019 Down.
	125	A	23.75	"	"	"	32800	32100	0.0078 S.	0.0076 Up.
	128		23.72	"	"	"	32400		0.0496 N.	0.0042 Up.
										
	155		23.69	"	17'-5$\frac{15}{16}$"	85	27200		0.3028 N.	0.0061 Up.
6A	158	A	23.73	"	"	"	26800	26800	0.3341 N.	0.0312 Down.
	159	1-I 10"×35-lb.=10.29□"	23.72	"	"	"	26400		0.3341 S.	0.0522 Down.
		2 Pls. 11"×$\frac{5}{8}$"=13.75								
	129	Total=24.04□"	23.64	"	24'-8⅜"	120	24900		0.1827 S.	0.0097 Up.
	130		23.83	"	"	"	24000	24800	0.4100 S.	0.0157 Down.
	131		23.76	"	"	"	25500		0.3654 N.	0.0099 Up.
	3	Light Sections.	10.77	2.33	9'-8½"	50	36100		0.06 S.	0.055 Up.
	4	A	10.80	"	"	"	35700	35800	0.05 N.	0.0251 Up.
	53		10.86	"	"	"	35600		0.0118 N.	0.0217 Up.
										
	19		10.78	"	16'-6$\frac{1}{16}$"	85	32500		0.29 S.	0.200 Up.
10	83	3½" A	10.76	"	"	"	31900	32100	0.0239 N.	0.1117 Down.
	86	2 Pls. 9"×¼"=4.500□"	10.76	"	"	"	31900		0.2611 N.	0.0786 Down.
		4 Ls. 2"×2"×¼"=3.760								
	23	2 Pls. 5¼"×¼"=2.625	10.75	"	23'-3⅝"	120	28400		0.30 N.	0.05 Down
	33	Total=10.885□"	10.76	"	"	"	28300	28400	0.23 S.	0.03 Up.
	34		10.75	"	"	"	28400		0.20 S.	0.00
	104	Heavy Sections.	18.60	2.34	9'-9"	50	31400		0.0360 S.	0.1556 Down.
	167	A	18.52	"	"	"	32000	31800	0.0099 N.	0.0851 Down.
	170		18.46	"	"	"	32100		0.0783 N.	0.0188 Down.
										
	144		18.67	"	16'-6⅞"	85	28000		0.0292 S.	0.1044 Down.
10A	149	3½" A	18.65	"	"	"	28000	28300	0.1545 S.	0.0992 Down.
	157	2 Pls. 9"×$\frac{7}{16}$"=7.88□"	18.69	"	"	"	29000		0.0731 N.	0.0010 Down.
		4 Ls. 2"×2"×$\frac{7}{16}$"=6.24								
	140	2 Pls. 5¼"×$\frac{7}{16}$"=4.59	18.64	"	23'-4$\frac{13}{16}$"	120	27000		0.0392 N.	0.1080 Up.
	141	Total=18.71□"	18.80	"	"	"	25900	26300	0.2260 S.	0.1112 Up.
	142		18.65	"	"	"	26000		0.2297 S.	0.1113 Up.

TABLE 8.

ABSTRACT OF RECORD OF COLUMN TESTS.
TYPES 7 & 7A – 8 & 8A

Type	Test no.	Section.	Actual Area	Radius of Gyration	Length.	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
7	5	. Light Sections	13.33	2.48	10'-4"	50	34300		0.00	0.03 Down.
	37		13.30	"	"	"	32700	33400	0.02 S.	0.04 Up.
	76		13.33	"	"	"	33200		0.0034 N.	0.1628 Down.
	15.		13.40	"	17'-6 13/16"	85	34000		0.10 S.	0.40 Up.
	89		13.43	"	"	"	30000	31600	0.1081 N.	0.2923 Up.
	90	4-5" Bulb L's x 10.1-lb = 11.88 □"	13.29	"	"	"	30700		0.0914 N.	0.1670 Up.
		1 Pl. 6" x 5/16" = 1.88								
	28	Total = 13.76 □"	13.36	"	24'-9 5/8"	120	28300		0.05 S.	0.18 Up.
	48		13.34	"	"	"	26900	28100	0.0137 N.	0.3020 Down.
	51		13.33	"	"	"	29000		0.0020 N.	0.0160 Up.
7A*		Heavy Sections.								
										
		4-5" Bulb L's x 13.50-lb = 15.88 □"								
		1 Pl. 6" x 7/16" = 2.63								
		Total = 18.51 □"								
8	9	Light Sections.	10.98	2.46	10'-3"	50	35900		0.00	0.03 Down.
	39		11.01	"	"	"	35100	35700	0.06 S.	0.06 Up.
	78		11.03	"	"	"	36200		0.0072 S.	0.0086 Up.
	16		11.09	"	17'-5 1/8"	85	31400		0.03 S.	0.11 Up.
	81		11.05	"	"	"	32900	32800	0.0324 N.	0.0653 Up.
	82	4-4" Z's x 1/4" = 9.64 □"	11.11	"	"	"	34000		0.0117 S.	0.1597 Up.
		1 Pl. 7" x 1/4" = 1.75								
	27	Total = 11.39 □"	11.06	"	24'-7 3/16"	120	29100		0.07 N.	0.06 Down.
	46		11.11	"	"	"	29400	29700	0.0417	0.0194 Up.
	50		11.16	"	"	"	30400		0.0088 N.	0.1281 Down.
8A	100	Heavy Sections.	25.78	2.51	10'-5 1/2"	50	33000		0.0235 N.	0.0431 Down.
	126		25.86	"	"	"	33500	32900	0.0313 N.	0.0940 Down.
	127		25.83	"	"	"	32200		0.0261 N.	0.2400 Down.
	156		25.56	"	17'-9 3/8"	85	31300		0.0835 S.	0.2297 Up.
	160		25.57	"	"	"	25300	29400	0.0010 S.	0.3393 Up.
	161	4-4" Z's x 5/8" = 22.20 □"	25.43	"	"	"	31500		0.0459 N.	0.1462 Up.
		1 Pl. 7" x 5/8" = 4.38								
	162	Total = 26.58 □"	25.66	"	25'-1 3/16"	120	27500		0.1019 S.	0.3075 Up.
	163		25.70	"	"	"	27000	27300	0.0848 N.	0.2025 Down.
	164		25.68	"	"	"	27500		0.0032 N.	0.760 Down.

* Tests have not been made.

COLUMN TEST RECORD
Form 200 a.

DEPARTMENT OF COMMERCE, BUREAU OF STANDARDS, WASHINGTON, D. C., REPORT ON TEST No. 30

Date, April 3 and 4, 1914.

For whom tested: American Society of Civil Engineers.
Type No. or designation: 1.
Marks on column: 1 A G 5451.
Tested with flat ends.

Counterweighted: one-half its weight at middle.
Nominal sectional area, in square inches: 11.475.
Radius of gyration: 2.24.
Slenderness ratio: 120.

Initial condition—
Riveting: Good.
Members at the ends: Good.
Alignment: Good (See note).

Length over all: 22 ft. 4 11/16 in.
Weight in pounds: 896.
Sectional area, in square inches: 11.36. Corrected area, 11.20.
Gauged lengths: 150 and 8 in.

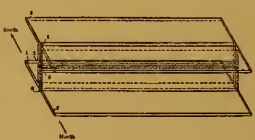
Applied Loads.		Compressions, in Inches, in 150-in. Gauged Lengths for Positions.					Mean compression to 150-in. gauged lengths (inches).	Deflections at Middle of Length (Inches).	
Pounds, total.	*Pounds per square inch.	1	2	3	4	5		Horizontal North.	Vertical Down.
11 360	1 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	....	......
56 800	5 000	0.0214	0.0212	0.0196	0.0205	0.0211	0.0206	0.00	0.00
11 360	1 000	0.0002	0.0005	0.0009	0.0004	0.0001	0.0001	....	......
56 800	5 000	0.0214	0.0212	0.0196	0.0206	0.0211	0.0209		
113 600	10 000	0.0470	0.0465	0.0459	0.0471	0.0461	0.0465	0.00	0.00
11 360	1 000	0.0020	0.0021	0.0004	0.0024	0.0004	0.0011	....	......
56 800	5 000	0.0224	0.0215	0.0191	0.0225	0.0156	0.0208		
113 600	10 000	0.0469	0.0475	0.0449	0.0476	0.0448	0.0463		
170 400	15 000	0.0722	0.0742	0.0716	0.0747	0.0720	0.0729	0.00	0.00
11 360	1 000	0.0022	0.0027	0.0016	0.0034	0.0011	0.0011	....	......
11 360	1 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	....	......
56 800	5 000	0.0210	0.0208	0.0210	0.0210	0.0213	0.0210		
113 600	10 000	0.0463	0.0468	0.0478	0.0473	0.0471	0.0486		
170 400	15 000	0.0717	0.0728	0.0784	0.0782	0.0789	0.0729		
181 750	16 000	0.0766	0.0788	0.0789	0.0787	0.0780	0.0784	0.00	0.00
193 120	17 000	0.0812	0.0828	0.0841	0.0840	0.0848	0.0833	0.00	0.006 D
204 480	18 000	0.0880	0.0880	0.0892	0.0897	0.0901	0.0886	0.00	0.01
215 840	19 000	0.0911	0.0934	0.0949	0.0968	0.0960	0.0942	0.00	0.01
227 200	20 000	0.0958	0.1006	0.1007	0.1014	0.1021	0.1001	0.00	0.02
11 360	1 000	0.0003	0.0015	0.0010	0.0026	0.0013	0.0013	0.00	0.006
56 800	5 000	0.0205	0.0212	0.0209	0.0223	0.0210	0.0212		
113 600	10 000	0.0460	0.0473	0.0469	0.0488	0.0473	0.0479		
170 400	15 000	0.0712	0.0738	0.0784	0.0750	0.0744	0.0735		
227 200	20 000	0.0965	0.0999	0.1006	0.1016	0.1017	0.1000		
238 660	21 000	0.1018	0.1066	0.1069	0.1077	0.1026	0.1056	0.00	0.02
249 920	22 000	0.1058	0.1121	0.1123	0.1141	0.1143	0.1117	0.00	0.02
261 280	23 000	0.1196	0.1184	0.1183	0.1212	0.1210	0.1160	0.00	0.03
272 640	24 000	0.1151	0.1254	0.1267	0.1285	0.1298	0.1250	0.00	0.04
284 000	25 000	0.1188	0.1315	0.1354	0.1369	0.1368	0.1319	0.00	0.06
11 660	1 000	0.0012	0.0057	0.0072	0.0095	0.0076	0.0069	0.00	0.06
56 600	5 000	0.0197	0.0278	0.0288	0.0299	0.0293	0.0270		
113 600	10 000	0.0448	0.0628	0.0660	0.0658	0.0658	0.0630		
170 400	15 000	0.0696	0.0793	0.0819	0.0829	0.0830	0.0793		
227 200	20 000	0.0944	0.1066	0.1089	0.1156	0.1121	0.1068		
284 000	25 000	0.1161	0.1338	0.1374	0.1381	0.1396	0.1330		
318 700	28 055	......	......	......	......	......	......	0.30 N	1.00 D
318 700	28 455	Ultimate Strength According to Corrected Area.						....	......

Applied Loads.		40 in. from Base. Compression, in Inches, in 8-in. Gauged Lengths at Base for Positions.							
Pounds, total.	*Pounds per square inch.	1	2	3	4	5	6	7	8
11 360	1 000	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	0.0000
56 800	5 000	0.0013	......	0.0013	0.0010	0.0011	......	0.0009	0.0011
11 360	1 000	0.0001	......	0.0001	0.0004	0.0001	......	0.0002	0.0000
56 800	5 000								
113 600	10 000	0.0027	......	0.0027	0.0023	0.0026	......	0.0025	0.0025
11 360	1 000	0.0001	......	0.0000	0.0004	0.0002	......	0.0001	0.0001
56 800	5 000								
113 600	10 000								
170 400	15 000	0.0042	......	0.0043	0.0037	0.0040	......	0.0040	0.0031
11 360	1 000	0.0001	......	0.0000	0.0004	0.0002	......	0.0001	0.0001
11 360	1 000	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	0.0000
56 800	5 000								
113 600	10 000								
170 400	15 000								
181 750	16 000	0.0044	......	0.0043	0.0044	0.0043	......	0.0044	0.0043
193 120	17 000	0.0047	......	0.0048	0.0048	0.0046	......	0.0047	0.0046
204 480	18 000	0.0060	......	0.0052	0.0052	0.0047	......	0.0060	0.0049
215 840	19 000	0.0063	......	0.0055	0.0055	0.0061	......	0.0054	0.0052
227 200	20 000	0.0065	......	0.0058	0.0060	0.0064	......	0.0058	0.0064
11 360	1 000	0.0001	......	0.0001	0.0003	0.0001	......	0.0000	0.0000
56 800	5 000								
113 600	10 000								
170 400	15 000								
227 200	20 000								
238 660	21 000	0.0069	......	0.0061	0.0063	0.0058	......	0.0061	0.0059
249 920	22 000	0.0062	......	0.0055	0.0068	0.0061	......	0.0064	0.0061
261 280	23 000	0.0065	......	0.0070	0.0072	0.0064	......	0.0068	0.0065
272 640	24 000	0.0067	......	0.0072	0.0070	0.0058	......	0.0072	0.0070
284 000	25 000	0.0071	......	0.0077	0.0080	0.0070	......	0.0076	0.0074
11 660	1 000	0.0001	......	0.0006	0.0010	0.0001	......	0.0002	0.0002
56 600	5 000								
113 600	10 000								
170 400	15 000								
227 200	20 000								
284 000	25 000								
318 700	28 055	Failed primarily by buckling of legs of angles, but finally by buckling in center. Temperature at finish 20.5° cent.							
318 700	28 455								

Applied Loads.		Straddling Middle. Compression, in Inches, in 8-in. Gauged Lengths at Middle for Positions.								Remarks.
Pounds, total.	*Pounds per square inch.	1	2	3	4	5	6	7	8	
11 360	1 000	0.0000	......	0.0000	......	0.0000	......	0.0000	0.0000	Temp. 20.9° cent.
56 800	5 000	0.0013	......	0.0012	......	0.0014	......	0.0013	0.0014	Note: 42 in. from top of column, bad kink in leg of angle along line of gaugings.
11 360	1 000	0.0001	......	0.0000	......	0.0000	......	0.0001	0.0002	No. 1 leg of angle bent in 0.6 in. The behavior of the angle is shown below.
56 800	5 000									
113 600	10 000	0.0026	......	0.0026	......	0.0026	......	0.0026	0.0029	
11 360	1 000	0.0001	......	0.0000	......	0.0000	......	0.0002	0.0004	
56 800	5 000									
113 600	10 000									
170 400	15 000	0.0041	......	0.0041	......	0.0043	......	0.0043	0.0044	Temp. 21° cent.
11 360	1 000	0.0001	......	0.0000	......	0.0000	......	0.0004	0.0004	
11 360	1 000	0.0000	......	0.0000	......	0.0000	......	0.0000	0.0000	April 4th. Temp. 19.5° cent.
56 800	5 000									
113 600	10 000									
170 400	15 000									
181 750	16 000	0.0044	......	0.0044	......	0.0046	......	0.0044	0.0046	Snapping noise.
193 120	17 000	0.0047	......	0.0049	......	0.0048	......	0.0046	0.0049	
204 480	18 000	0.0050	......	0.0052	......	0.0051	......	0.0048	0.0052	
215 840	19 000	0.0052	......	0.0056	......	0.0055	......	0.0051	0.0055	Snapping noise.
227 200	20 000	0.0064	......	0.0059	......	0.0068	......	0.0068	0.0060	Slight drop of beam.
11 360	1 000	0.0000	......	0.0002	......	0.0000	......	0.0001	0.0008	Slight loosening of scale on angles.
56 800	5 000									
113 600	10 000									
170 400	15 000									
227 200	20 000									
238 660	21 000	0.0068	......	0.0062	......	0.0063	......	0.0069	0.0062	
249 920	22 000	0.0061	......	0.0068	......	0.0066	......	0.0062	0.0068	
261 280	23 000	0.0064	......	0.0072	......	0.0070	......	0.0068	0.0069	Snapping noise.
272 640	24 000	0.0068	......	0.0076	......	0.0074	......	0.0068	0.0072	
284 000	25 000	0.0068	......	0.0086	......	0.0078	......	0.0071	0.0076	Legs of angles bending.
11 660	1 000	0.0000	......	0.0011	......	0.0002	......	0.0008	0.0008	Scaling pretty general on 5-in. legs of angles.

Applied Loads.		40 in. from Top. Compression, in Inches, in 8-in. Gauged Lengths at Top for Positions.							
Pounds, total.	*Pounds per square inch.	1	2	3	4	5	6	7	8
11 360	1 000	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	0.0000
56 600	5 000	0.0011	......	0.0009	0.0008	0.0012	......	0.0013	0.0012
11 360	1 000	0.0000	......	0.0001	0.0001	0.0000	......	0.0001	0.0002
56 800	5 000	......	......	......	......	......	......	......	......
113 600	10 000	0.0024	......	0.0022	0.0023	0.0026	......	0.0026	0.0030
11 660	1 000	0.0001	......	0.0001	0.0001	0.0000	......	0.0001	0.0000
56 800	5 000	......	......	......	......	......	......	......	......
113 600	10 000	......	......	......	......	......	......	......	......
170 400	15 000	0.0035	......	0.0040	0.0068	0.0058	......	0.0037	0.0050
11 360	1 000	0.0002	......	0.0000	0.0002	0.0000	......	0.0002	0.0001
11 660	1 000	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	0.0000
56 800	5 000	......	......	......	......	......	......	......	......
113 000	10 000	......	......	......	......	......	......	......	......
170 400	15 000	......	......	......	......	......	......	......	......
181 760	16 000	0.0042	......	0.0043	0.0042	0.0040	......	0.0041	0.0063
193 120	17 000	0.0044	......	0.0043	0.0046	0.0044	......	0.0043	0.0064
204 480	18 000	0.0047	......	0.0050	0.0048	0.0047	......	0.0045	0.0071
215 840	19 000	0.0050	......	0.0054	0.0048	0.0049	......	0.0046	0.0076
227 200	20 000	0.0053	......	0.0068	0.0052	0.0050	......	0.0049	0.0082
11 360	1 000	0.0000	......	0.0004	0.0000	0.0000	......	0.0004	0.0019
56 600	5 000	......	......	......	......	......	......	......	......
113 000	10 000	......	......	......	......	......	......	......	......
170 400	15 000	......	......	......	......	......	......	......	......
227 200	20 000	......	......	......	......	......	......	......	......
238 660	21 000	0.0056	......	0.0062	0.0065	0.0063	......	0.0040	0.0087
249 920	22 000	0.0059	......	0.0068	0.0067	0.0065	......	0.0049	0.0094
261 280	23 000	0.0062	......	0.0072	0.0060	0.0067	......	0.0061	0.0103
272 640	24 000	0.0064	......	0.0077	0.0062	0.0068	......	0.0043	0.0113
284 000	25 000	0.0068	......	0.0082	0.0064	0.0058	......	0.0046	0.0124
11 660	1 000	0.0001	......	0.0071	0.0004	0.0004	......	0.0017	0.0044

Applied Loads.		Compression, in Inches, in ......Gauged Lengths.								Variations in Distances Between Legs of Angles, at Bend in Flanges.								
Pounds, total.	*Pounds per square inch.	1	2	3	4	5	6	7	8	Start of bend.	Max. bend or center.	Start of bend.	South side. 1/4	South side. 1/2	South side. 3/4	North side. 1/4	North side. 1/2	North side. 3/4
11 360	1 000	......	......	......	......	......	......	......	......	5.592	5.016	5.581						
56 600	5 000	......	......	......	......	......	......	......	......	5.591	5.008	5.585	0.000	......	0.000	0.000	......	0.002 open.
11 360	1 000	......	......	......	......	......	......	......	......	5.589	5.015	5.561						
56 800	5 000	......	......	......	......	......	......	......	......	......	......	......						
113 600	10 000	......	......	......	......	......	......	......	......	5.600	4.993	5.588	......	......	......	......	......	0.004 open.
11 660	1 000	......	......	......	......	......	......	......	......	5.593	4.018	5.581						
56 800	5 000	......	......	......	......	......	......	......	......	5.006	4.985	5.594	......	......	......	......	......	0.006 open.
113 600	10 000	......	......	......	......	......	......	......	......	......	......	......						
170 400	15 000	......	......	......	......	......	......	......	......	6.064	4.011	5.583						
11 360	1 000	......	......	......	......	......	......	......	......	......	......	......						
11 660	1 000	......	......	......	......	......	......	......	......	......	......	......						
56 800	5 000	......	......	......	......	......	......	......	......	......	......	......						
113 000	10 000	......	......	......	......	......	......	......	......	......	......	......						
170 400	15 000	......	......	......	......	......	......	......	......	......	......	......						
181 760	16 000	......	......	......	......	......	......	......	......	5.606	4.974	5.595						
193 120	17 000	......	......	......	......	......	......	......	......	5.607	4.971	5.596						
204 480	18 000	......	......	......	......	......	......	......	......	......	......							
215 840	19 000	......	......	......	......	......	......	......	......	......	......							
227 200	20 000	......	......	......	......	......	......	......	......	......	......							
11 360	1 000	......	......	......	......	......	......	......	......	......	5.24							
56 600	5 000	......	......	......	......	......	......	......	......	......	......							
113 000	10 000	......	......	......	......	......	......	......	......	......	......							
170 400	15 000	......	......	......	......	......	......	......	......	......	......							
227 200	20 000	......	......	......	......	......	......	......	......	......	5.19		......	......	......	......	......	0.01 open.
238 660	21 000	......	......	......	......	......	......	......	......	......	5.18							
249 920	22 000	......	......	......	......	......	......	......	......	......	5.17							
261 280	23 000	......	......	......	......	......	......	......	......	......	5.16							
272 640	24 000	......	......	......	......	......	......	......	......	......	5.14							
284 000	25 000	......	......	......	......	......	......	......	......	......	5.20							
11 660	1 000	......	......	......	......	......	......	......	......	......	5.01							



TEST No. 30.

* Sectional area used in computing unit loading, 11.36 sq. in. Corrected area, 11.20 sq. in. All loadings in pounds per square inch should be adjusted by multiplying by the factor, 1.0143.

DETAILS OF TEST No. 30, TO ACCOMPANY PROGRESS REPORT OF SPECIAL COMMITTEE ON STEEL COLUMNS AND STRUTS.

forms to discover the effect of variability of shape on the strength of the column. Nine of these types have been made and tested, and the tenth one, which was a circular form, has been omitted, owing to the fact that, thus far, it has not been practicable to obtain a circular column with the same high quality of material which has been provided for the rest of the series.

Types 1, 2, 3, and 10 are commercial riveted columns, most commonly used at the present time. Type 5 has met with much favor in building work. It had been suggested to the Committee that failure may frequently be due to the fact that thin outstanding legs cripple before the main section of the column is seriously affected by the load, and therefore Types 4, 7, and 8 were selected to determine the effect of a square corner or bulb on the edge of the outstanding legs. Study seems to show very little difference in the results, so far as the types are concerned, but when we consider the effect of light and heavy sections, there is a marked difference in favor of the thin material. The variation in this rule appears in the columns, slenderness ratio $50\ \frac{l}{r}$, of Type 6, which is an I-shape with two thin plates riveted to the flanges of the I-beam. The Committee realized that such arrangement was not considered good standard practice, but desired to determine the effect of thin outstanding edges.

The general falling off of the unit ultimate for heavy sections carries through all the other types of columns, and it will be noted that this is also true for the Bethlehem H-columns, which are solid rolled sections, as well as for the riveted sections.

Your Committee would say that up to the time of the making of this report, its studies have not progressed sufficiently to draw a conclusion in regard to the falling off of the strength of the heavy material. For the benefit of those who desire to study the problem, Table 9 shows the chemical and physical specimen tensile tests for the columns of Type 1 and Type 1-*A*.

The study of the stress-strain curves for Tests 84-*A*, 84-*B*, 85-*A*, 98-*A*, 98-*B*, and 99-*A*, which are shown on Fig. 1, serves to throw some additional light on the problem. These test columns, it will be remembered, were cut from longer columns having the same test number, so as to give short columns with a slenderness ratio of $20\ \frac{l}{r}$. It will be noted that though the heavy sections of these short columns had a unit ultimate strength considerably higher than that of the light sections, the stress-strain curve indicates that the yield point of the heavy sections is lower than that of the light sections. These columns were tested in accordance with the regular programme of loadings up to 5 000, 10 000, 15 000, 20 000, 25 000, and 30 000 lb. per sq. in.,

TABLE 9.—CHEMICAL AND PHYSICAL SPECIMEN TENSILE TESTS.

Columns, Type 1; Light Section.

Section.	Heat No.	Yield point.	Ultimate strength.	Carbon.	Manganese.	Phosphorus.	Sulphur.
6 × 5/16 webs.	11 105	34 450 32 950 35 480* 36 950*	60 260 59 730 59 680 60 860	0.20	0.47	0.010	0.038
5 × 3 × 5/16 angles.	42 171	34 270 34 070	57 990 57 740	0.20	0.44	0.010	0.028
Average....		34 695	59 380				

* Slow-speed test made with aid of magnifying glass by Bureau of Standards.

Columns, Type 1-*A*; Heavy Section.

Section.	Heat No.	Yield point.	Ultimate strength.	Carbon.	Manganese.	Phosphorus.	Sulphur.
6 × 5/8 webs...	38 054	36 940 36 790	58 400 58 100	0.20	0.40	0.020	0.032
5 × 3 × 5/8 angles.	13 354	38 280 38 800	61 430 60 920	0.20	0.40	0.010	0.031
	1 257	38 300 39 100	60 460 58 970	0.22	0.50	0.015	0.030
	10 032	38 280 38 240	57 800 58 760	0.20	0.46	0.019	0.034
Average....		38 090	59 355				

dropping back in each case to the initial load of 1 000 lb. per sq. in., and, as part of the original long columns, they had previously been subjected to the same programme of loading.

The curves plotted are all for the seventh application, or run-up, of the load. In the case of the light sections, the yield points for the seventh run-up are well above 30 000 lb., and as the sixth run-up and the ultimate on the original Column No. 85 only reached this load, it is probable that the yield points were not materially raised by previous run-ups.

In the case of the heavy sections, however, the yield points, even after the seventh run-up, were below 30 000 lb., and had been exceeded on the sixth run-up. There is little doubt, therefore, that the original yield point for these sections was less than that shown by the seventh run-up, having been artificially raised by the sixth run-up. Until tests can be made on short columns, with a slenderness ratio of 20 $\frac{l}{r}$,

PLATE LXXIV.
PAPERS, AM. SOC. C. E.
DECEMBER, 1915.
PROGRESS REPORT OF
SPECIAL COMMITTEE ON
COLUMNS AND STRUTS.

COLUMN TEST RECORD
Form 260 a.

DEPARTMENT OF COMMERCE, BUREAU OF STANDARDS, WASHINGTON, D. C.—REPORT ON TEST No. 54.

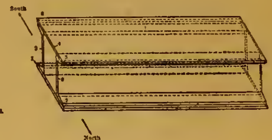
Date, July 24, 1914.

For whom tested: American Society of Civil Engineers.
Type No. or designation: 2.
Marks on column: 2A.
Tested with flat ends.

Counterweighted half its weight at middle.
Nominal sectional area, in square inches: 10.18.
Radius of gyration: 2.31.
Slenderness ratio: 120.

Initial condition—
Riveting: Good.
Members at the ends: Good.
Alignment: Vertical, good. Bent south 0.2 in. in center.

Length over all: 23 ft. 1$\frac{1}{16}$ in.
Weight in pounds: 840.
Sectional area, in square inches: 9.66.
Gauged lengths: 150 and 8 in. Corrected area, 10.09 sq. in.

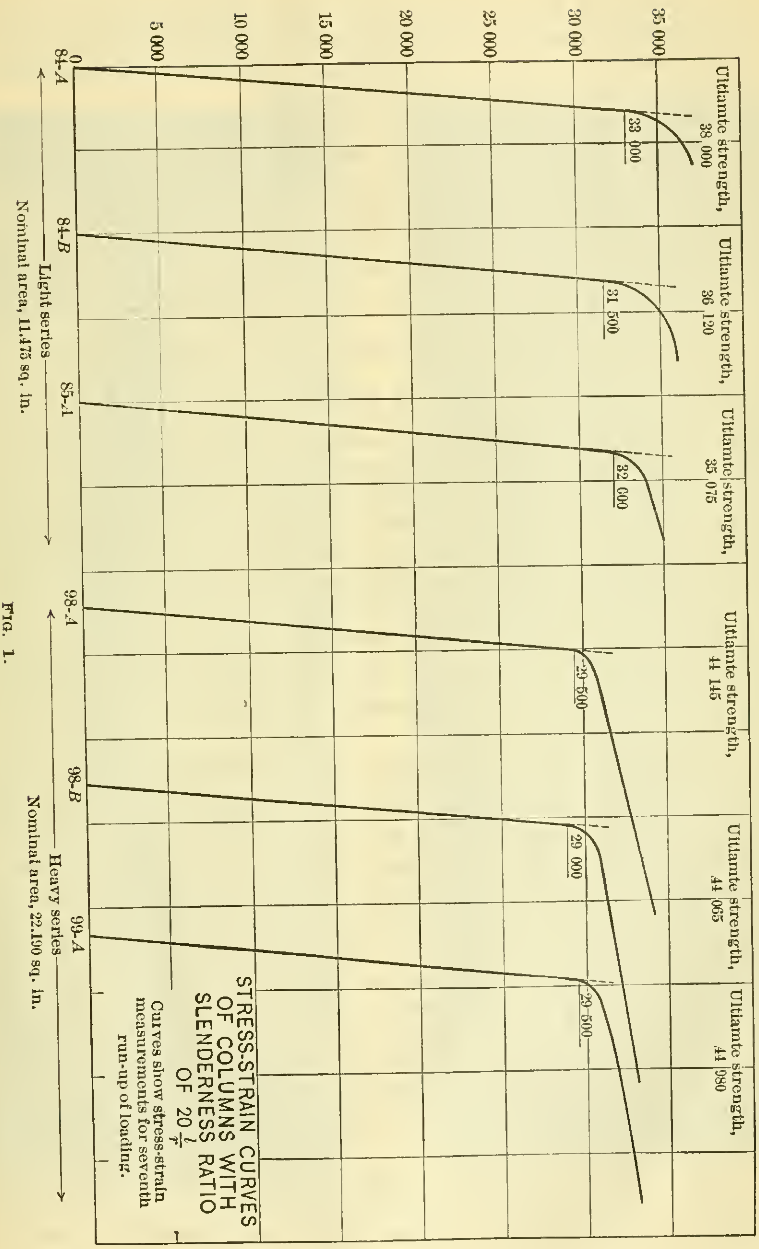


TEST No. 54.

APPLIED LOADS.		COMPRESSIONS, IN INCHES, IN 150-IN. GAUGED LENGTHS FOR POSITIONS.					Mean compression in 150-in. gauged lengths (inches).	DEFLECTIONS AT MIDDLE OF LENGTH (INCHES).		STRADDLING MIDDLE. COMPRESSION, IN INCHES, IN 8-IN. GAUGED LENGTHS AT MIDDLE FOR POSITIONS.								Remarks.
Pounds, total.	*Pounds per square inch.	1	3	5	7	4		Horizontal South.	Vertical Up.	1	2	3	4	5	6	7	8	
9 660	1 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	......	0.0000	0.0000	0.0000	0.0000	
48 600	5 000	0.0197	0.0183	0.0204	0.0210	0.0198	0.0198	0.0097	0.0097	0.0012	0.0011	0.0010	......	0.0012	0.0011	0.0011	0.0012	
9 660	1 000	0.0006	0.0002	0.0001	0.0002	0.0003	0.0001	0.0005	0.0134	0.0000	0.0000	0.0000	......	0.0001	0.0000	0.0000	0.0001	
48 600	5 000	0.0196	0.0183	0.0202	0.0210	0.0212	0.0201											
96 600	10 000	0.0440	0.0430	0.0464	0.0477	0.0459	0.0452	0.0234	0.0172	0.0025	0.0024	0.0023	......	0.0026	0.0028	0.0027	0.0025	
9 660	1 000	0.0011	0.0003	0.0004	0.0000	0.0020	0.0008	0.0016	0.0143	0.0000	0.0001	0.0000	......	0.0001	0.0000	0.0000	0.0001	
48 300	5 000	0.0203	0.0188	0.0209	0.0265	0.0217	0.0216											
96 600	10 000	0.0442	0.0422	0.0469	0.0478	0.0463	0.0465											
144 900	15 000	0.0681	0.0656	0.0728	0.0745	0.0705	0.0704	0.0403	0.0214	0.0036	0.0035	0.0035	......	0.0041	0.0039	0.0041	0.0036	
9 660	1 000	0.0013	0.0005	0.0004	0.0014	0.0023	0.0012	0.0030	0.0166	0.0000	0.0000	0.0000	......	0.0002	0.0001	0.0003	0.0002	
48 300	5 000	0.0209	0.0193	0.0216	0.0226	0.0225	0.0214											
96 600	10 000	0.0446	0.0427	0.0472	0.0486	0.0469	0.0460											
144 900	15 000	0.0683	0.0657	0.0729	0.0747	0.0713	0.0706											
154 660	16 000	0.0728	0.0702	0.0784	0.0800	0.0763	0.0755	0.0444	0.0229	0.0040	0.0038	0.0037	......	0.0044	0.0042	0.0044	0.0042	
164 220	17 000	0.0778	0.0747	0.0836	0.0853	0.0810	0.0805	0.0484	0.0235	0.0043	0.0041	0.0039	......	0.0048	0.0046	0.0048	0.0045	Snapping noise.
173 880	18 000	0.0825	0.0793	0.0889	0.0906	0.0859	0.0855	0.0527	0.0245	0.0044	0.0044	0.0043	......	0.0051	0.0048	0.0050	0.0048	
183 540	19 000	0.0873	0.0836	0.0940	0.0962	0.0908	0.0904	0.0578	0.0259	0.0047	0.0046	0.0044	......	0.0053	0.0051	0.0054	0.0049	
193 200	20 000	0.0921	0.0883	0.0999	0.1020	0.0957	0.0956	0.0630	0.0272	0.0050	0.0050	0.0048	......	0.0057	0.0054	0.0057	0.0053	
9 660	1 000	0.0018	0.0008	0.0011	0.0023	0.0028	0.0017	0.0061	0.0174	0.0001	0.0002	0.0001	......	0.0001	0.0000	0.0001	0.0001	
48 300	5 000	0.0210	0.0192	0.0219	0.0233	0.0228	0.0216	0.0162	0.0162									
96 600	10 000	0.0450	0.0429	0.0481	0.0498	0.0473	0.0466	0.0297	0.0205									
144 000	15 000	0.0687	0.0653	0.0739	0.0755	0.0721	0.0711	0.0460	0.0242									
193 200	20 000	0.0923	0.0887	0.1000	0.1021	0.0962	0.0959	0.0637	0.0282									
193 200	20 000	0.0923	0.0889	0.1006	0.1022	0.0961	0.0960	......	......	......	......	......	......	......	......	......	......	After 5 min.
193 200	20 000	0.0924	0.0891	0.1004	0.1022	0.0961	0.0960	......	......	......	......	......	......	......	......	......	......	After 10 min.
193 200	20 000	0.0924	0.0892	0.1004	0.1022	0.0961	0.0961	0.0640	0.0283	......	......	......	......	......	......	......	......	After 15 min.
202 860	21 000	0.0970	0.0934	0.1055	0.1077	0.1010	0.1009	0.0684	0.0289	0.0052	0.0052	0.0050	......	0.0061	0.0058	0.0061	0.0056	
212 520	22 000	0.1016	0.0981	0.1111	0.1135	0.1062	0.1061	0.0752	0.0307	0.0056	0.0054	0.0052	......	0.0064	0.0061	0.0064	0.0059	
222 180	23 000	0.1064	0.1025	0.1169	0.1192	0.1112	0.1112	0.0817	0.0324	0.0057	0.0056	0.0054	......	0.0066	0.0064	0.0066	0.0063	Snapping noise
231 840	24 000	0.1118	0.1075	0.1231	0.1260	0.1168	0.1170	0.0906	0.0348	0.0059	0.0060	0.0058	......	0.0070	0.0069	0.0072	0.0067	" "
241 600	25 000	0.1169	0.1121	0.1294	0.1327	0.1230	0.1226	0.0998	0.0378	0.0061	0.0062	0.0066	......	0.0076	0.0072	0.0077	0.0071	
9 600	1 000	0.0039	0.0029	0.0039	0.0068	0.0049	0.0043	0.0156	0.0226	0.0001	0.0001	0.0001	......	0.0001	0.0000	0.0002	0.0001	
48 800	5 000	0.0230	0.0209	0.0249	0.0272	0.0245	0.0241	0.0266	0.0204									
96 600	10 000	0.0470	0.0441	0.0512	0.0540	0.0493	0.0491	0.0407	0.0257									
144 900	15 000	0.0709	0.0677	0.0769	0.0800	0.0740	0.0739	0.0576	0.0300									
193 200	20 000	0.0943	0.0900	0.1033	0.1072	0.0986	0.0987	0.0783	0.0344									
241 500	25 000	0.1175	0.1124	0.1300	0.1337	0.1225	0.1232	0.1026	0.0385	......	......	......	......	......	......	......	......	Scale lines on top cover-plate 36 in. and 43 in. from base.
241 600	25 000	0.1176	0.1127	0.1304	0.1338	0.1226	0.1234	......	......	......	......	......	......	......	......	......	......	After 5 min.
241 600	25 000	0.1178	0.1128	0.1304	0.1341	0.1227	0.1235	......	......	......	......	......	......	......	......	......	......	After 10 min.
241 600	25 000	0.1178	0.1128	0.1306	0.1342	0.1228	0.1236	......	......	......	......	......	......	......	......	......	......	After 15 min.
241 500	25 000	0.1179	0.1130	0.1306	0.1344	0.1230	0.1238	0.1041	0.0389	......	......	......	......	......	......	......	......	After 20 min.
241 600	25 000	0.1180	0.1130	0.1309	0.1344	0.1231	0.1239	0.1041	0.0392	......	......	......	......	......	......	......	......	After 25 min.
9 000	1 000	0.0044	0.0028	0.0044	0.0068	0.0052	0.0047	0.0171	0.0234	......	......	......	......	......	......	......	......	July 24th.
9 660	1 000	0.0030	0.0016	0.0029	0.0052	0.0038	0.0033	0.0171	0.0181	......	......	......	......	......	......	......	......	" 25th.
48 300	5 000	0.0224	0.0202	0.0236	0.0265	0.0236	0.0233	0.0281	0.0204									
96 600	10 000	0.0462	0.0455	0.0500	0.0528	0.0488	0.0482	0.0418	0.0258	......	......	......	......	......	......	......	......	Temperature: 27½° cent.
144 900	15 000	0.0701	0.0670	0.0759	0.0794	0.0729	0.0731	0.0691	0.0298									
193 200	20 000	0.0934	0.0897	0.1021	0.1060	0.0974	0.0977	0.0694	0.0344									
241 600	25 000	0.1167	0.1120	0.1292	0.1334	0.1204	0.1223	0.1042	0.0386									
241 600	25 000	0.1167	0.1119	0.1292	0.1334	0.1206	0.1224	0.1043	0.0385	......	......	......	......	......	......	......	......	After 5 min. Scale lines on top cover-plate not any more developed.
251 160	26 000	0.1212	0.1163	0.1348	0.1590	0.1253	0.1274	0.1102	0.0396	0.0066	0.0067	0.0062	......	0.0066	0.0078	0.0066	0.0078	
251 160	26 000	0.1214	0.1104	0.1649	0.1390	0.1255	0.1274	0.1111	0.0401	......	......	......	......	......	......	......	......	After 5 min.
251 160	26 000	0.1213	0.1136	0.1349	0.1390	0.1254	0.1274	......	......	......	......	......	......	......	......	......	......	After 10 min. Scaling a little more prominent.
251 160	26 000	0.1214	0.1164	0.1351	0.1393	0.1255	0.1276	0.1127	0.0401	......	......	......	......	......	......	......	......	After 15 min.
251 160	26 000	0.1214	0.1162	0.1354	0.1395	0.1258	0.1277	0.1133	0.0401	......	......	......	......	......	......	......	......	After 20 min.
251 160	26 000	0.1216	0.1167	0.1354	0.1396	0.1258	0.1278	0.1135	0.0401	......	......	......	......	......	......	......	......	After 25 min.
251 160	26 000	0.1218	0.1167	0.1354	0.1398	0.1260	0.1279	......	......	......	......	......	......	......	......	......	......	After 30 min.
251 160	26 000	0.1218	0.1167	0.1354	0.1398	0.1260	0.1279	......	......	......	......	......	......	......	......	......	......	After 35 min.
260 820	27 000	0.1264	0.1208	0.1431	0.1480	0.1317	0.1340	0.1331	0.0431	0.0066	0.0069	0.0063	......	0.0066	0.0084	0.0092	0.0080	
260 820	27 000	0.1264	0.1208	0.1464	0.1481	0.1300	0.1311	......	......	......	......	......	......	......	......	......	......	After 5 min.
260 820	27 000	0.1265	0.1208	0.1442	0.1485	0.1322	0.1344	0.1371	0.0436	......	......	......	......	......	......	......	......	After 10 min.
260 820	27 000	0.1266	0.1209	0.1439	0.1487	0.1324	0.1315	......	......	......	......	......	......	......	......	......	......	After 15 min.
260 820	27 000	0.1267	0.1210	0.1442	0.1490	0.1327	0.1347	0.1403	0.0438	......	......	......	......	......	......	......	......	After 20 min.
260 820	27 000	0.1266	0.1205	0.1441	0.1493	0.1327	0.1347	......	......	......	......	......	......	......	......	......	......	After 25 min.
260 820	27 000	0.1267	0.1207	0.1446	0.1493	0.1329	0.1348	0.1421	0.0441	......	......	......	......	......	......	......	......	After 30 min.
270 480	28 000	0.1308	0.1247	0.1514	0.1586	0.1382	0.1407	0.1709	0.0477	0.0069	0.0072	0.0063	......	0.0092	0.0090	0.0099	0.0065	
270 480	28 000	0.1311	0.1243	0.1551	0.1608	0.1395	0.1422	0.1869	0.0495	......	......	......	......	......	......	......	......	After 5 min.
270 480	28 000	0.1312	0.1241	0.1562	0.1620	0.1400	0.1427	0.1910	0.0503	......	......	......	......	......	......	......	......	After 10 min.
270 480	28 000	0.1309	0.1239	0.1565	0.1623	0.1400	0.1427	0.1946	0.0506	......	......	......	......	......	......	......	......	After 15 min.
270 480	28 000	0.1311	0.1239	0.1569	0.1628	0.1403	0.1430	0.1970	0.0508	......	......	......	......	......	......	......	......	After 20 min.
270 480	28 000	0.1311	0.1239	0.1572	0.1632	0.1404	0.1432	0.1996	0.0509	......	......	......	......	......	......	......	......	After 25 min.
270 480	28 000	0.1310	0.1239	0.1576	0.1637	0.1404	0.1433	0.2010	0.0509	......	......	......	......	......	......	......	......	After 30 min.
270 480	28 000	0.1311	0.1239	0.1581	0.1638	0.1407	0.1435	0.2022	0.0509	......	......	......	......	......	......	......	......	After 35 min.
270 480	28 000	0.1313	0.1239	0.1582	0.1638	0.1408	0.1436	0.2030	0.0509	......	......	......	......	......	......	......	......	After 40 min.
270 480	28 000	0.1313	0.1239	0.1584	0.1640	0.1410	0.1437	0.2041	0.0509	......	......	......	......	......	......	......	......	After 45 min.
270 480	28 000	0.1313	0.1239	0.1584	0.1640	0.1411	0.1437	0.2050	0.0509	......	......	......	......	......	......	......	......	After 50 min.
270 480	28 000	0.1315	0.1240	0.1585	0.1643	0.1412	0.1439	0.2058	0.0509	......	......	......	......	......	......	......	......	After 55 min.
270 480	28 000	0.1316	0.1240	0.1589	0.1645	0.1413	0.1441	0.2063	0.0509	......	......	......	......	......	......	......	......	After 60 min.
270 480	28 000	0.1316	0.1241	0.1589	0.1648	0.1418	0.1441	0.2070	0.0509	......	......	......	......	......	......	......	......	After 65 min.
270 480	28 000	0.1316	0.1241	0.1594	0.1652	0.1414	0.1443	0.2072	0.0509	......	......	......	......	......	......	......	......	After 70 min.
280 140	29 000	0.1353	0.1266	0.1689	0.1790	0.1482	0.1510	0.2750	0.0583	0.0068	0.0073	0.0060	......	0.0105	0.0101	0.0116	0.0092	Scale lines at about 18 in. spacing on web of chan. on N. side.
280 140	29 000	0.1340	0.1245	0.1764	0.1842	0.1500	0.1538	0.3060	0.0616	......	......	......	......	......	......	......	......	After 5 min.
280 140	29 000	0.1338	0.1238	0.1798	0.1865	0.1507	0.1547	0.3237	0.0626	......	......	......	......	......	......	......	......	After 10 min.
280 140	29 000	0.1333	0.1232	0.1802	0.1883	0.1512	0.1552	0.3350	0.0643	......	......	......	......	......	......	......	......	After 15 min.
280 140	29 000	0.1333	0.1228	0.1819	0.1898	0.1518	0.1559	0.3437	0.0638	......	......	......	......	......	......	......	......	After 20 min.
280 140	29 000	0.1331	0.1225	0.1826	0.1910	0.1521	0.1563	0.3521	0.0642	......	......	......	......	......	......	......	......	After 25 min.
280 140	29 000	0.1332	0.1225	0.1839	0.1922	0.1523	0.1568	0.3595	0.0650	......	......	......	......	......	......	......	......	After 30 min.
280 140	29 000	0.1331	0.1225	0.1844	0.1927	0.1527	0.1571	0.3640	0.0655	......	......	......	......	......	......	......	......	After 35 min.
280 140	29 000	0.1331	0.1223	0.1850	0.1930	0.1528	0.1572	0.3670	0.0657	......	......	......	......	......	......	......	......	After 40 min.
280 140	29 000	0.1331	0.1222	0.1854	0.1935	0.1528	0.1574	0.3706	0.0662	......	......	......	......	......	......	......	......	After 45 min.
280 140	29 000	0.1331	0.1220	0.1854	0.1935	0.1529	0.1574	0.3717	0.0664	......	......	......	......	......	......	......	......	After 50 min.
289 800	30 000	0.1110	0.0925	0.2494	0.2658	0.1709	0.1779	0.9950	0.0586									
289 800	28 720	Ultimate Strength According to Corrected Area.																

*Sectional area used in computing unit loading, 9.66 sq. in. Corrected area, 10.09 sq. in. All loadings in pounds per square inch should be adjusted by multiplying by the factor, 0.9574.

DETAILS OF TEST No. 54, TO ACCOMPANY PROGRESS REPORT OF SPECIAL COMMITTEE ON STEEL COLUMNS AND STRUTS.



0
5 000
10 000
15 000
20 000
25 000
30 000
35 000
Ultlamte strength, 38 000
33 000
Ultlamte strength, 36 120
31 500
Ultlamte strength, 35 075
32 000
Ultlamte strength, 44 145
29 500
Ultlamte strength, 44 065
29 000
Ultlamte strength, 44 980
29 500
84-A
84-B
85-A
98-A
98-B
99-A
Light series
Nominal area, 11.475 sq. in.
Heavy series
Nominal area, 22.190 sq. in.
STRESS-STRAIN CURVES OF COLUMNS WITH SLENDERNESS RATIO OF $20\frac{l}{r}$
Curves show stress-strain measurements for seventh run-up of loading.

FIG. 1.

which have not been previously stressed, your Committee does not care to do any more than draw attention to the yield points indicated by the stress-strain curves, with the thought that this information may indicate the way toward the correct solution.

The Bureau of Standards, at the time the material for the original columns was ordered, arranged to have the rolling mill furnish 5-ft. length pieces from each melt number, for special physical and chemical tests. In order that the subject may have careful investigation, the Bureau is now making a series of re-tests on the steel from each melt number. These tests are to be made on short specimen sections, in compression as well as in tension, and, it is to be hoped, will throw additional light on the problem.

A matter of interesting comparison is that of the Z-bar columns, Type 8, slenderness ratio 50 $\frac{l}{r}$, as shown in the half-tones of typical light and heavy sections, Figs. 2 and 3. It will be noted that the light sections gave results comparing favorably with other types, but that in failure there was a tendency for the outstanding legs of the Z-bars to twist to a marked extent. The heavy section, Z-bar columns indicate the same falling off of unit ultimate which is shown by the other types, but it will be seen by Figs. 2 and 3 that the tendency to twist is not shown in these heavy Z-bars.

It will be seen from the foregoing tables that there has been no attempt to indicate a yield point for the columns with a slenderness ratio of 50 to 120 $\frac{l}{r}$. It is true that the stress-strain curves shown on Fig. 1 for the columns with a slenderness ratio of 20 $\frac{l}{r}$ very clearly indicate a yield point, and your Committee has given this question careful study, to see if such a yield point can be accurately determined for the longer columns. In making the tests, the compressions are carried to unit loadings of 5 000, 10 000, 15 000, 20 000, and 25 000 lb., and are then dropped back to the initial load of 1 000 lb. From the study of this series of tests, it would seem that initial sets occurred at very small loadings, and that these initial sets could be measured, provided the instruments could be obtained precise enough for the purpose. It would seem from the tests thus far studied that the yield point for the short columns is appreciably below the ultimate strength, but that for longer columns, in which bending action is shown, the yield point and the ultimate are probably very close together.

Thus far, your Committee has been engaged in the practical investigation of the details of the column tests, and has not yet had opportunity to co-ordinate these practical results with theoretical investigations.

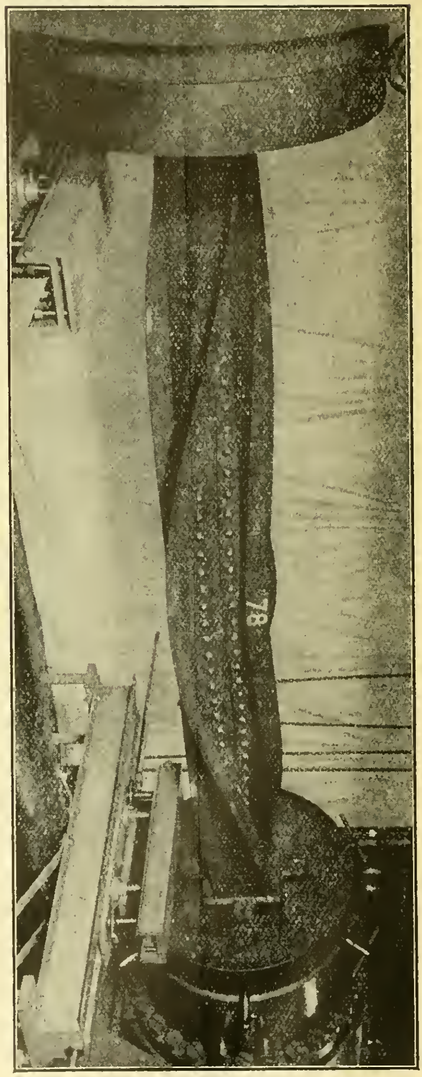


FIG. 2.—Z-BAR COLUMN. TYPE 8. LIGHT SECTION.

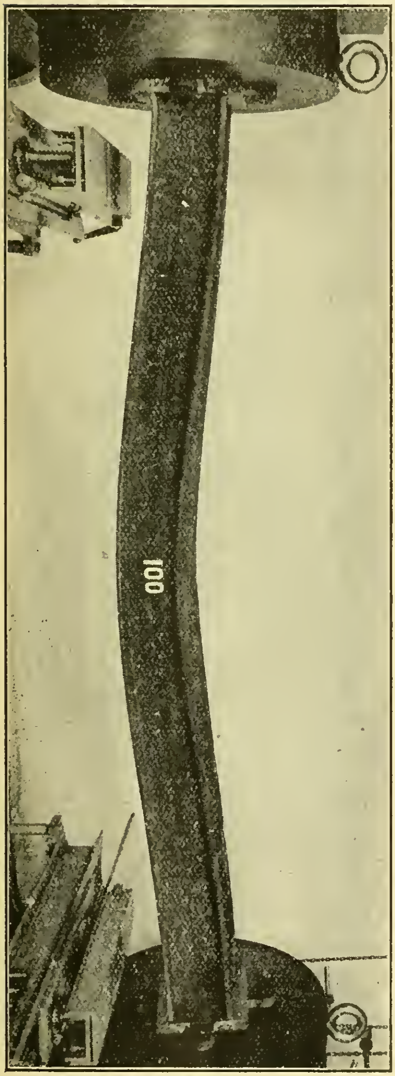


FIG. 3.—Z-BAR COLUMN. TYPE 8. HEAVY SECTION.

Your Committee desires at this time to express its appreciation of the cordial assistance which it has received from the Bureau of Standards, Dr. S. W. Stratton, Director, and the Engineer-Physicist, Dr. G. R. Olshausen. The members will no doubt be interested to know that the material for the original and supplementary programmes has been furnished through the Bureau, at the expense of the United States Government. Dr. Stratton has placed Dr. Olshausen and a corps of assistants in charge of this work, and a large part of their time has been devoted to it.

In view of the fact that additional tests are still to be made, your Committee presents this report of progress.

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Chairman.
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Secretary.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

PROGRESS REPORT OF THE SPECIAL COMMITTEE ON FLOODS AND FLOOD PREVENTION*

General.—With the growth of the United States, the question of flood control is daily assuming greater importance, but as yet has not received the attention which it merits. When a country is a wilderness, a flood occasions little damage, and is rarely recorded, but as the valleys are cleared and the farmer attempts to utilize them, the injurious effects of floods begin to be felt. With the growth of cities and villages, there arises a still greater demand for protection against overflow. While there is no conclusive evidence that great floods have increased in frequency or volume in recent years, with the growth and development of the country, the damage which is caused has been greatly intensified, and the obstruction of the flood plain by the works of man has in many cases largely increased flood heights for the same volume of discharge.

It is only in recent years that the property losses during floods have been reported, but these are rapidly increasing and becoming a serious burden on the resources of the country. Nor are they confined to any particular section. Floods of great intensity are liable to occur in semi-arid regions where the mean annual rainfall is less than 20 in., though possibly with less frequency than in regions having an average rainfall of over 60 in.; and a mountain stream with a slope of 6 ft. to the mile, is as liable to overflow its banks as an alluvial river with a slope of 6 in. per mile. In fact, the greatest loss of life from floods usually occurs on the minor streams, whose waters rise with great rapidity and flow with great velocity.

Data Deficient.—On most streams the physical data collected have been for other purposes than flood control, and are therefore deficient in the specific information necessary for such an investigation. In a river which is being improved for navigation, the essential element requiring consideration is the low-water flow.

* To be presented to the Annual Meeting, January 19th, 1916.

When a stream is to be utilized for water-power, again the first consideration is the low-water flow. If it is proposed to increase the low-water discharge by means of reservoirs, the engineer is interested in average conditions, and if he discusses those of extreme floods, it is only to provide a sufficient capacity for flood discharge without danger. Similarly, in determining the water-supply for a city, it is the low and mean flow of the stream which receives special consideration.

In discussing flood control, low and average conditions are of little value. A great flood is caused by an exceptional combination of most adverse conditions. A mean rain falling on a soil having a mean percolation and mean run-off, with a mean evaporation and other average conditions, produces about a mid-stage flow in a river. The extreme flood stage occurs when a maximum precipitation falls on a soil which because of frost or previous saturation has a minimum percolation and a maximum run-off, with a minimum evaporation. Such combinations are comparatively rare, and unless some one is specially delegated to investigate them, generally escape observation.

It is not only the causes of extreme flood discharge, however, on which additional data and further study are required. There is a large field for investigation in the influence of the works of man on the regimen of streams, and in the evaluation of all the little-understood factors that determine the flood height obtained from any given discharge.

Suggested Methods.—The methods of flood control which have been suggested may be summarized as follows:

(*a*) Methods of reducing extreme variations in rainfall and increasing the soil absorption of water, so as to diminish the run-off; reforestation, soil drainage, variations in methods of plowing, the construction of small dikes in agricultural areas, which will retain on the soil a large portion of the rainfall until there is time to absorb it.

(*b*) Reservoir and detention basins which will retain the crests of the run-off and discharge them at lower stages.

(*c*) Barriers thrown across mountain streams to prevent erosion and the transportation of débris.

(*d*) Channel improvement, cut-offs, and the construction of auxiliary channels.

(*e*) Levees which will confine the flood waters to the channel.

(*f*) Outlets for the purpose of facilitating the discharge of the streams.

Present Knowledge.—There is nothing to indicate that man can modify either the intensity or distribution of precipitation. To utilize the forces of Nature, which will reduce the effects of excessive

precipitation or increase the absorptive power of soils, necessitates an intimate knowledge of the intensity and distribution of the rainfall over the drainage area to be considered, of the character of its vegetation, of the permeability and depth of the overlying soil, and of the capacity of subterranean fissures and conduits to carry off the water which may penetrate to the underlying formations—subjects on which but a limited amount of information is at present available to the engineer.

The Weather Bureau has established stations for observing the precipitation, and the results are published in its monthly bulletins. Most of these observations, however, extend over a relatively short period of time, and it is improbable that the maximum precipitation that may occur at a given locality has yet been recorded. Moreover, they are generally taken in cities on high buildings where the gauges are not protected from wind, or in villages located in the valleys, and the relation of the precipitation over a built-up territory to that of the surrounding country and upon the uplands and divides has not been well determined.

The series of studies made by the United States Geological Survey on the surface water supply of the United States are necessarily based on comparatively few observations frequently taken at less than flood stages. With such an extensive field to cover, the observers cannot wait for extreme conditions on each stream, but must pass from stream to stream measuring the discharge which happens to exist when the observer is present. In 1914 the Geological Survey made stream-flow measurements at 3 400 points and maintained 1 480 gauges in conjunction with co-operating parties; many more are needed.

Care must be exercised in the use of discharge curves or rating tables. They may represent only average conditions, and when applied to a particular case may lead to large errors. Their extension above a bank-full stage cannot be depended upon for accurate results. With sedimentary streams in particular, both the slope and the area of cross-section are subject to abrupt changes, and floating débris may land against a pier or otherwise obstruct the flow in such a manner as to entirely change the relations which exist at lower stages.

At every town subject to overflow from a neighboring stream, there is urgent need of co-operation, not only to extend the discharge curves to extreme stages, but also to investigate the variations which may occur. On rivers which are being improved for navigation, such co-operation may be afforded by those engaged in the work of river improvement, and even though not strictly required for the development of a navigable channel, should, if necessary, receive Congressional authorization. On other streams, State commissions and private agencies are at present collecting data, all of which should be co-ordinated, studied, and published by some National agency.

But in considering flood control, the amount and character of the sediment carried is next in importance to the discharge of the stream. The advisability of reservoir and barrier construction, the practicability of enlarging the stream section, of affording relief by auxiliary channels, and of levee control, will be influenced by this factor. In streams carrying a large amount of sediment, the checking of the normal velocity by reservoir construction will cause its deposit; a sudden enlargement of the section of a stream will similarly reduce velocities and cause deposits, and this deposit is liable to occur during low stages, filling the channel on which reliance is placed for relief from flood discharge. If an auxiliary channel is built, a difference in velocity in the two channels is accompanied by a tendency of one or both to fill.

An extensive series of observations on the flow of sediment in the Mississippi and Missouri Rivers has been made by officers of the Corps of Engineers, U. S. Army, and the Mississippi River Commission, from 1879 to 1884. The Geological Survey has also published, in Water Supply Papers 236 and 274, investigations of the flow of sediment in other rivers.

Similar data on many other streams requiring flood control are lacking, and should be supplied.

A knowledge of the topography of the country, which is essential before plans for flood control can be formulated, is also lacking. In the Thirty-Fourth Annual Report of the Geological Survey it is stated that about 38.9% of the United States had been surveyed, and these surveys are generally plotted on maps of a scale of 1 : 62 500 and 1 : 125 000 (about 1 in. to 1 and 2 miles, respectively) with contours 20 and 50 ft. apart. Such surveying and mapping, even in greater detail, should be prosecuted with vigor and co-operative action by the General Government and the various States until the entire country has been covered.

For the large rivers the present maps give but few contours at wide intervals in the valleys, and the usefulness of such data in a study of flood control is therefore limited to purposes of reconnaissance. With the exception of those of the Mississippi, Missouri, and Illinois Rivers, the surveys for the improvement of navigation have been generally confined to the river bed.

The organic act creating the Mississippi River Commission stipulated among other duties that such surveys, examinations, and investigations of the Mississippi River and its tributaries should be made as might be deemed necessary for the purpose of formulating plans for the prevention of destructive floods. As a result of this legislation the Mississippi River Commission has been engaged for the last 35 years in collecting data relating to the flood control of the Mississippi River. An accurate hydrographic survey has been made of the river

from its head-waters to its mouth, with a topographical survey covering practically the entire width of the valley down to the head of the alluvial basin near the mouth of the Ohio, and extending to a distance of about 1 mile upon either side, with occasional sections across the entire alluvial valley, below the Ohio. Maps have been prepared on a scale of 1: 10 000 with contours 5 ft. apart. Discharge stations have been established on the main river and also on some of the larger tributaries where the flow is not affected by back-water, and the discharge curves have been developed from a long series of observations. In the earlier stages of the work, daily observations of sediment and discharge were made at several stations for periods of over a year. In later years, discharge measurements have been confined to prescribed stages above the normal high water and below the average low water, in order to trace any changes in regimen that might occur. Numerous gauge stations have also been established on the main river and its principal tributaries where the height of the river is recorded at least daily, and observations have been made to determine the slope, at both high and low water, and other hydraulic factors.

By the River and Harbor Act of June 18th, 1880, a project was adopted for the construction of forty-one reservoirs in Minnesota and Wisconsin to collect the surplus water from the precipitation of winter, spring, and early summer, and release it during low water so as to benefit navigation on the Mississippi River. In 1887 the scope of the project was reduced to include reservoirs at the head-waters of the rivers in Minnesota, only, and there have been constructed six reservoirs with a capacity of about 96 000 000 000 cu. ft. and having a drainage area of 4 535 sq. miles.

These reservoirs, while built for the low-water regulation of the river, were much larger than any others that had then been built by man for the high- or low-water control of rivers.* Careful records have been kept of the rainfall, the contents, and discharge of these reservoirs since their construction; and they afford most valuable data as to the flood control of a river by reservoirs, particularly as there is a different relation for each reservoir between its capacity and the area of its water-shed.

The surveys and estimates for reservoirs in Wisconsin, under the original project, afford further data for the discussion of flood control of the tributaries of the Mississippi River in that State.

The California Débris Commission has carried on a series of investigations of the Sacramento River and its tributaries, which are useful in determining the flood control of that drainage area. Important studies on the Allegheny and Monongahela drainage area have been made by the Pittsburgh Flood Commission. A large amount of

* Three basins on the Ottawa River now under construction will have a capacity of 168 000 000 000 cu. ft.

data on the Miami River has been accumulated, in connection with the work of the Dayton Flood Prevention Committee. The Scioto River has been made the subject of an extensive study at Columbus. And several flood commissions have been appointed by Governmental agencies, such as the Ohio River Flood Board of the U. S. Army Engineers and the Commission of the States of Indiana and Pennsylvania, though few of these have yet had sufficient opportunity to contribute any extensive studies upon stream regulation. Some of the minor valleys have been surveyed by cities to determine the proper source of a water supply, and a few corporations have determined the capacity of certain streams to develop water power; but, on the whole, there is a great dearth of reliable data as to rainfall, run-off, and other physical facts needed by the engineer for the proper solution of the problem of flood control, and the accumulation of the proper data will require many years of careful observation. Even with proper data available, the determination of the method of flood control, which is to be applied to a particular case, will require careful consideration and the greatest exercise of engineering skill. No method can be devised that will be susceptible of universal application.

Discussion of Suggested Methods.

(*a*) *Reforestation.*—If reforestation is considered merely from a commercial standpoint, the value to the country of reproducing our timber is so great that it obtains general approval, particularly of engineers who appreciate more than any other class the disadvantages to which the country will be subjected by the destruction of its forests.

But some advocates of reforestation have claimed that the destruction of our forests has increased the height and frequency of floods and diminished the discharge during low water, and have based their claim for reforestation on the beneficial influence of forests in preventing floods and improving the navigation of our rivers.

The relation of forest to stream flow has been extensively discussed in this country, in papers of recent date, by the U. S. Geological Survey; U. S. Weather Bureau; Flood Board of U. S. Engineers, on the Ohio River; Pittsburgh Flood Commission; Norton, on the Rivers of Michigan; Chittenden, before the American Society of Civil Engineers; Moore; Burr, on Merrimac River; Mead, on Rivers of Wisconsin; and the Second Report of the National Commission for the Conservation of the Waters of the United States; and others. In Europe, in addition to the classic papers of Gustave Wex, in 1873 and 1879, Keller, Lauda, and Oppokow have recently discussed conditions in Germany, Austria, and Russia, and a paper by G. Fantoli, 1913, sums up the status in the valley of the Po. An interesting discussion of the subject will also be found in the *Proceedings* of the International Navigation Congress in Milan, 1905.

In these discussions the greatest diversity of opinion has been expressed, and even the advocates of reforestation as a means of flood control fail to give any quantitative determination of the effects of forests upon floods. For the engineer to utilize any agency practically, he must know that it will produce a positive effect on which he can rely and which he can measure. The data as to the effect of forestation, soil absorption, and kindred methods of flood control are not at present susceptible of quantitative analysis.

Over a large portion of the United States freezing occurs during winter, and a heavy precipitation may occur when the ground is covered with an impenetrable layer of ice and much of the vegetation has shed its leaves. Such conditions prevent the utilization of forestation or ground storage. This was the principal cause of the great flood of 1912 which devastated the Ohio and Mississippi Valleys.

(*b*) *Reservoirs.*—The equalizing effect of lake storage on the discharge of the St. Lawrence and Oswego Rivers is frequently quoted as illustrating efficient flood control by reservoirs. But in these cases Nature has been much more lavish in the expenditure of its resources than man can afford to be. About one-third of the water-shed of the Great Lakes is devoted to reservoir purposes.

The permanent overflowing of land which can be devoted to agriculture is costly, and reservoir construction is most economical when confined to mountainous valleys where the area of tillable land is small when compared with the volume of water that can be stored. On many streams this condition materially limits the applicability of reservoirs as a means of flood control, particularly in thickly settled districts.

It also frequently happens that a reservoir can be more profitably utilized for other purposes, such as power development, irrigation, or the water supply of a city. There is a popular delusion that the same reservoir can be utilized simultaneously to reduce floods, increase the low-water discharge of a stream, and increase the water-power that can be developed therefrom, but ordinarily its utilization for any one of these purposes precludes its efficient use for either of the others. If a reservoir is to be utilized to diminish floods, it is essential that it be empty when a heavy rain occurs. If to increase the low-water discharge of a river, it must be full when the rainy season ceases. To insure the first condition, it is necessary to empty the reservoir after every storm, to obtain space to store the discharge of the one following. To insure the second, it is necessary to close the outlets of the reservoir at relatively low stages, and when it is once filled, not permit the surplus water to escape until the river falls to the stage when the stored water is required for navigation. Both of these processes interfere with the development of water-power, whose effectiveness is measured by the minimum discharge permitted. The cause of the

trouble is the irregularity of rainfall in different years, and the necessity of being prepared for a minimum rainfall when considering navigation interests and a maximum for controlling flood conditions. In years of average rainfall, it may be practicable to utilize a reservoir for different purposes, but it is reiterated that average conditions do not enter into the problem of flood control.

Your Committee, however, does not intend to condemn *in toto* the utilization of reservoirs for more than one purpose. In fact, it believes that the practical solution of the flood problem in some valleys will be found in permitting corporations to build reservoirs in which a portion of the stored water can be utilized to a limited extent for power purposes and the remainder for flood prevention, but in such cases one of the conditions of the franchise must be the construction of reservoirs far in excess of the dimensions required for the development of the proposed power, and a rigid supervision must be exercised by the authority granting the franchise, not only to see that its requirements are fulfilled in construction, but also in operation.

There are in some localities opportunities for arresting the flow of a stream during the progress of a flood by temporary storage which limits the river discharge to that which may be safely confined within banks. This can be done by detention basins which differ from storage reservoirs in that they do not require the abandonment of the sites occupied for storage purposes. Flood control in such cases is only necessary during exceptional rains, and the basin itself may therefore be devoted to its normal uses most of the time, and only an easement on the lands occupied is necessary.

By this means cities can be protected at a minimum inconvenience to the rural community. The use of detention basins has been adopted by the Miami Conservancy District for the purpose of controlling the floods of the Miami Basin.

(*c*) *Barriers.*—Barriers erected across the valleys of tributary streams have been utilized to retain detritus and prevent its deposition in the main river. When conditions are favorable, this is a practical method of control.

(*d*) *Channel Enlargement and Cut-Offs.*—A stream flowing in an alluvial valley adjusts its channel to its discharge; when the discharge increases, the area of its cross-section is increased, and when the discharge diminishes the area of cross-section is reduced. In a river like the Missouri or the Colorado, carrying a large amount of sediment, these changes are rapid.

Where a channel is enlarged for the purpose of carrying extreme floods, it should be so designed as to give a velocity at all stages, which will prevent deposits. If not so designed, the periodic removal of deposits by mechanical means will be necessary.

A cut-off or straightening of an alluvial channel affects its slope which the stream attempts to regain, producing excessive caving of banks. Both stream-enlargement and cut-offs, while reducing flood heights immediately above them, increase the discharge capacity locally and increase flood heights below them. In alluvial rivers such work may seriously affect the regimen of the low-water channel, producing shoals, both above and below them, and should not be attempted when questions of navigation are involved or bank erosion may seriously affect other interests.

In streams of stable banks, and beds carrying but little sediment, such straightening may not be injurious.

(*e*) *Levees.*—Every alluvial stream flowing in a valley which has been formed by its own deposits, builds up its banks to a certain height which is generally known as the bank-full stage. Below that stage the stream flows in a more or less clearly defined trough and, at higher stages, it spreads over the alluvial valley. When an unleveed river rises above the bank-full stage there is an abrupt change in its regimen. The area of its cross-section increases very rapidly and ceasing to follow the sinuosities which its low-water channel has created, its currents tend to flow across points, materially changing the slope. These changes in regimen affect the movement of sediment, causing deposits at some localities and scour at others.

When the river is leveed this abrupt change in regimen is prevented to a greater or less extent, dependent on the characteristics of the river and the distance of the levee from the river bank. The function of a levee, as far as river hydraulics are concerned, is to extend the natural banks up to extreme flood stages and thus continue or possibly intensify the conditions which exist at a bank-full stage, *i. e.*, wherever a stream tends to scour or fill at a bank-full stage, that tendency may be somewhat intensified at extreme flood stages if the stream be controlled by levees which have been properly located with reference to its banks.

There is another feature in connection with levees which it is believed has been generally overlooked, and that is the increase of channel storage which they create, and the resultant reduction of flood height from this cause. Under present conditions of levee alignment, between the mouths of the Ohio and Red Rivers, the Mississippi River has a storage capacity of 1 365 000 000 000 cu. ft. (about 30 000 000 acre-ft.).* When we consider this enormous reservoir effect, it is evident that flood control on the Mississippi River below the Ohio cannot be realized without the use of adequate levees.

(*f*) *Outlets.*—Outlets are at best of doubtful utility in reducing the flood heights of sedimentary streams, even where additional lines of flow to the sea or lake into which the stream empties are practicable.

* The Roosevelt Dam in Arizona stores about 1 500 000 acre-ft.

The frictional resistance of the river bed limits their influence to a comparatively short distance above them. In a delta formation they usually require leveeing to protect the adjacent country from overflow and in a river carrying a high percentage of sediment, either the outlet or the main stream will diminish its area of cross-section by fill and thus reduce the discharge capacity and ultimately defeat the purpose for which the outlet is constructed.

Three natural outlets from the Mississippi River to the Gulf of Mexico may be cited as illustrating this principle. The Atchafalaya River leaves the Mississippi at a point about 200 miles above New Orleans and reaches tide-water at a distance of 100 miles from its source, while the main river traverses a distance of 300 miles before reaching the Gulf. Although having three times the slope of the main river, the entrance to this outlet practically closed itself at low stages by natural processes of deposit and, in order to maintain navigable depths, annual dredging has become necessary, and if this were suspended the closure would doubtless in time become effective.

The Jump, an outlet which leaves the right bank of the river about 10 miles above the Head of the Passes, was once a formidable outlet, but is now of insignificant proportions; Cubits Gap on the left bank, about 5 miles above the Head of the Passes, which was cut through to the Gulf in 1862, at first enlarged very rapidly to a width of over ½ mile with a maximum depth of 80 ft. near the entrance, but has now become a shallow outlet which is being rapidly obliterated by deposits.

So far as the Mississippi River is concerned, whose regimen has been closely studied for over half a century, the observed effects of the natural outlets that have been described and the effects of crevasses in levees carrying one-fourth of the flood volume of the river with but a small reduction in gauge heights, all point to the conclusion that the use of outlets in any form for the purpose of reducing flood heights will generally fail to secure the desired results.

Conclusions.

Your Committee therefore submits the following conclusions:

Reforestation.—The effects of forest growth in preventing erosion on hillsides are sufficient to justify reforestation for that purpose, but there has been no quantitative determination of its influence on stream flow, which would justify its employment as a method of flood prevention.

Reservoirs and Detention Basins.—At the head-waters of streams, storage reservoirs and detention basins can be successfully employed to reduce flood heights. Which method is preferable is dependent on local conditions. Their efficiency, however, rapidly diminishes as the distance from them increases.

Cut-Offs, Channel Enlargement, By-Pass, and Outlets.—In clear-water streams, whose banks and beds are not subject to scour, these agencies may be profitably employed. In streams carrying a large amount of sediment, whose banks are readily eroded, great care should be exercised in their employment, always bearing in mind the fact that the best conditions on an alluvial stream follow its confinement to a single channel.

Levees.—As you proceed down stream the influence of reservoirs on flood prevention rapidly diminishes, and the influence of levees correspondingly increases in importance as a method of flood protection. On the lower alluvial reaches of long rivers, such as the Mississippi and Colorado, they afford the only sure means of flood control.

Your Committee believes that it can perform no greater service to the Profession than to call attention to the paucity of the data existing in reference to flood control and to the damage which may result from river regulation legislation, either by the Nation or the States, calling for definite projects which are not predicated on full and thorough investigation.

While valuable results have been obtained by National departments and bureaus and State and local commissions, with the limited means at their disposal, such investigations have generally been subordinated to other questions than flood control, and there is pressing need of intensive study directed especially to this end. Moreover, these agencies have worked on independent lines, and there has not been the necessary co-ordination of their observations. Progress would be much more rapid if each local investigation could be brought to follow some uniform method of procedure and to cover all of the points included in a standard outline, which would permit the fullest utilization of the data obtained.

Your Committee desires to urge the great importance of the early establishment and unification of systematic rainfall, run-off, and flood observations, covering the entire country in far greater detail than has yet been attempted.

While the co-operation of all agencies engaged in river regulation is desirable, the question of flood control is becoming of such vital importance to the country that it should not be subordinated to questions of navigation or power development. The disasters of 1913 in the Ohio Valley—involving a property loss of \$67 000 000, and more than 361 lives—and that at Kansas City, in 1903—involving a property damage of at least \$10 000 000—occurred on non-navigable streams, and on some of the navigable rivers the interests affected by floods are of far greater importance than those connected with navigation. If considered merely as an accessory to navigation, the limits of investigation of flood problems will be vastly curtailed, and their usefulness correspondingly diminished.

The members of your Committee have received reports from many sources covering investigations in connection with stream flow, and have been much gratified with the prompt, cordial responses to inquiries for data. In digesting this information it soon became apparent that reliable records of past floods were too meager and covered too brief a period to justify at this time a general report on the floods of the country.

A voluminous report might be prepared by compiling and tabulating areas of drainage basins, slopes of streams, gauge readings, oscillations in stage, and other physical data which the Committee has gathered. But this did not seem to be judicious, since only a small portion of our members are interested in such information and those who are would doubtless prefer to go to the official records for the more complete data required in their special work. Moreover, the expense and labor involved in preparing and printing such data would be disproportionate to the value of the results.

C. McD. Townsend, *Chairman,*
John A. Bensel,
T. G. Dabney,
J. B. Lippincott,
Daniel W. Mead,
J. A. Ockerson,
Arthur T. Safford,
Charles Saville,
F. L. Sellew,
C. E. Grunsky.*

*Endorsement of the Committee Report by C. E. Grunsky.

In concurring with the conclusions of the Committee on Floods and Flood Prevention, the undersigned desires to emphasize the fact that he is in favor of cut-offs and the straightening of river channels whenever conditions permit. The cut-off is not undesirable, as might be inferred from the language of the report, but desirable, and yet there will be cases where bank protection may be too expensive to justify recourse to the use of cut-offs that might otherwise be thought advisable.

He desires, too, to add that, after the flood flow has been reduced by all practicable means, the aim should be:

a.—To make the stream carry as much of the flood flow as can be carried between levees of reasonable height.

b.—To allow water in excess of the maximum stream capacity to escape from the river under control at selected points.

c.—To convey the water which is allowed to escape from a river during flood stages, to a re-entry into the river or to a suitable place of outfall, under such control as will prevent undue inundation.

C. E. Grunsky.

MINORITY REPORT.

By Morris Knowles, M. Am. Soc. C. E.

The writer regrets that at the last moment he finds himself in a position which requires that he submit a minority report. His disappointment is especially keen, as he was and is in substantial agreement with the report as originally drafted by the sub-committee. It is only because some portions of the report now presented represent so wide a departure therefrom and because of failure to secure such reconsideration of these changes by a reconvened committee meeting that he feels it incumbent upon himself to present these views for the consideration of the Society.

His objections to the report as now submitted relate to some expressions scattered through the paper, in which only a single view is presented upon controversial subjects upon which it must be recognized that much more information can and should be obtained. Such expressions, he believes, promote controversy rather than helpful discussion, and discourage rather than encourage further investigation. It would be far better to express—as is so ably done by our Board of Direction when submitting questions for vote—the reasons for and against various opinions. It is unwise to assume that we have at any time obtained all the knowledge possible upon a given subject. If we will approach the question from this point of view and with full confidence that the truth in the end must prevail, the opinions of the Society, when finally expressed, will deserve and receive recognition as authoritative.

For these reasons, therefore, the writer presents these amendments for consideration by the Society:

(1).—Page 2774, the third paragraph reads as follows:

> "Similar data on many other streams requiring flood control are lacking, and should be supplied."

In order that it may not appear that the kind of observations already made are entirely sufficient, which is probably not the case, and as there are several other items of data required for a complete study, the writer suggests that the following paragraph be substituted for that above quoted:

> "Extensive studies are needed regarding the physical and physiographic laws affecting stream loads, rates of transportation and deposition, and the variation of these factors with different conditions."

(2).—Page 2777, the first sentence, first paragraph, reads as follows:

> "In these discussions the greatest diversity of opinion has been expressed, and even the advocates of reforestation as a means of flood control fail to give any quantitative determination of the effects of forests upon floods."

This is one of the most controversial subjects upon which engineering and scientific minds have been recently engaged, and inductive reasoning does not at present result in conclusive proof in all cases upon either side of the dispute. The writer, therefore, suggests the following sentence as a substitute for that above quoted:

> "In these discussions the greatest diversity of opinion has been expressed and neither the advocates nor the opponents of reforestation as a means of flood control have completely established to universal satisfaction a quantitative determination of the effect of forests upon floods."

(3).—Page 2777, the third sentence, first paragraph, reads as follows:

> "The data as to the effect of forestation, soil absorption, and kindred methods of flood control are not at present susceptible of quantitative analysis."

Indicating our desire, as previously stated, to stimulate further research, the writer suggests the following substitute for the above quoted sentence:

> "Intensive study as to the effect of reforestation, soil absorption, and kindred methods of flood control is needed before quantitative analysis will be possible."

(4).—Page 2777, the second sentence, second paragraph, referring to frozen ground, reads as follows:

> "Such conditions prevent the utilization of forestation or ground storage."

As such conditions prevent such utilization at such times and such places, only, a more accurate statement is in the language recommended to be substituted for the above, *viz.*:

> "These conditions prevent at such times the utilization of reforestation or ground storage."

(5).—Page 2777, the second sentence, last paragraph, reads as follows:

> "There is a popular delusion that the same reservoir can be utilized simultaneously to reduce floods, increase the low-water discharge of a stream, and increase the water-power that can be developed therefrom, but ordinarily its utilization for any one of these purposes precludes its efficient use for either of the others."

As this indicates the kind of indirect adverse criticism which does not tell the whole story, the following is recommended as a substitute:

> "However, the same reservoir capacity cannot be used simultaneously to reduce floods, increase the low-water discharge of the stream, and increase the water-power that can be de-

veloped therefrom, but ordinarily, unless there be excess capacity, the use of the reservoir for any one of these purposes precludes its complete use for others."

(6).—The remainder of the paragraph just referred to comprises an entirely unnecessary and unfair discussion, purporting to show that reservoirs cannot be made useful for flood prevention, together with other purposes; whereas we know, on the contrary, that, notwithstanding all these statements, it is possible to operate reservoirs with several purposes in view, and that this is actually done in some of the great reservoir systems of Europe and America without conflict of interests. The following two sentences are recommended as a substitute:

> "It is evident, however, that such reduced efficiency for a combination of purposes does have some value, and the maximum efficiency for each purpose can be obtained by increasing the capacity sufficiently. The problem is an economic one, and much information regarding flood frequency, flood damage, reservoir operation, and comparative values of the several uses of storage is necessary before it can be satisfactorily solved in any case."

(7).—It is recommended that the first sentence in the second paragraph on page 2778, reading as follows, be omitted:

> "Your Committee, however, does not intend to condemn *in toto* the utilization of reservoirs for more than one purpose. In fact, it believes that"—

Such "damning with faint praise" is entirely unnecessary, and if such condemnation is not intended, it will be evident from the expressions, without saying so. The paragraph may well be introduced with the second sentence by omitting the first five words.

(8).—Page 2778, the third sentence, third paragraph, referring to the use of detention basins, reads as follows:

> "Flood control in such cases is only necessary during exceptional rains, and the basin itself may therefore be devoted to its normal uses most of the time, and only an easement on the lands occupied is necessary."

While it is understood that some agricultural pursuits can be carried on within and under the flow line of such detention basins, the use and occupancy of buildings, especially within their lower depth, likely to be filled to some extent each spring, cannot be permitted. Therefore the following is recommended as a more accurate statement, and as a substitute for the above quotation:

> "Complete utilization of the basin for such flood control is only necessary during exceptional rains and the land of the basin itself may therefore be devoted to its normal agricultural use most of the time and only the easement is necessary."

(9).—Page 2778, paragraph entitled (*c*): This seems to be a rather indefinite statement regarding barriers, and the following is recommended as a substitute:

> "Barriers erected across the valleys of tributary streams have been utilized to retain detritus and prevent its deposition in the main river. This has been found to be a suitable method in certain sections of our western country, where the wash of large amounts of material into the streams had disturbed the equilibrium and regimen. It is also used in Western Europe. Where conditions are favorable, this is a practical method of control, and its use will depend upon the slope of the stream and the character of the sediment being transported."

(10).—Page 2779, the fifth paragraph discusses the subject of increase of channel storage between levees. The writer believes that such storage is ineffective, in view of the fact that the potential storage over the surrounding country and overflowed lands is much greater than the volume confined between the levees. Therefore, considering any point on a stream below the length included within levees, the flood height is increased and not decreased by the introduction of the levees above. The writer, therefore, recommends that this paragraph be omitted.

(11).—For reasons which are apparent in the discussion hereinbefore given, the writer suggests that the four conclusions on pages 2780 and 2781 be amended to read as follows:

> "*Reforestation.*—The effects of forest growth in preventing erosion on hillsides are sufficient to justify reforestation for that purpose, but there has not been sufficient quantitative determination of its influence on stream flow which would justify a universal conclusion as to its employment as a method of flood control.
>
> "*Reservoirs and Detention Basins.*—At the head-waters of streams, storage reservoirs and detention basins can be successfully employed to reduce flood height. Local conditions will determine which method is preferable. The efficiency, however, of such agencies diminishes as the distance from them increases.
>
> "*Channel Enlargement, Cut-Offs, and Outlets.*—In clear-water streams, whose banks and beds are not subject to scour, these agencies are useful. In streams carrying a large amount of sediment, whose banks are readily eroded, great care should be exercised in their employment, always bearing in mind the fact that on an alluvial stream best conditions follow its confinement to a single channel.
>
> "*Levees.*—The influence of levees as a method of flood control becomes of prime importance on the lower alluvial reaches of long rivers and, upon certain streams, may afford the only method of flood control."

(12).—It is of little avail to submit such conclusions and recommendations without a suggestion as to some definite agency to carry on

such work. Such constructive criticism will be helpful and should be presented. The writer therefore suggests the omission of the fifth paragraph on page 2781 and the substitution of the following, which should probably occur at the conclusion of the entire report.

> "This subject is one of national importance, and while the work of State and local agencies should not be minimized, the necessity for the early establishment and unification, under National control, of systematic rainfall, run-off, and flood observations in greater detail than at present should be emphasized. Such authority should also suggest uniform methods of observation, co-ordinate information obtained, and preferably should have these duties as its main purpose, even though construction of important works for realization of results may be carried on at the same time by other agencies.
>
> "The Committee desires to urge the great importance of the early establishment of such special agency, supported by adequate appropriations, for the purpose of studying stream regulation in its largest sense and under whose direction all data shall be collated, according to uniform standards and systems, so that appropriate development of the science shall be made."

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

EDWARD GRAY, M. Am. Soc. C. E.*

DIED OCTOBER 2D, 1915.

Edward Gray, the son of Mr. and Mrs. James Gray, was born at Belfast, Ireland, on December 6th, 1875. When he was two years old, his parents came to the United States and settled in Princeton, Ind., where the boy was reared and educated in the public schools.

He began his professional career in 1896, when he entered the office of E. E. Watts, County Surveyor and City Engineer of Princeton. In 1897, Mr. Gray was appointed Deputy County Surveyor, and had charge, during 1897 and 1898, of dredging operations for irrigation, surveys for roads, and other municipal work.

In 1898, he was appointed Draftsman on the Louisville, Evansville, and St. Louis Consolidated Railway, and, in 1899, was made Assistant Engineer in charge of location and maintenance.

In 1901, Mr. Gray entered the employ of the Southern Railway Company as Assistant Engineer. He remained with the Company for 11 years, serving as Principal Assistant Engineer, Engineer of Maintenance of Way, and Chief Engineer of the St. Louis and Louisville Division, with headquarters at St. Louis, Mo.

In 1912, his health began to fail, and he went to Colorado where he became Chief Engineer of the Florence and Cripple Creek Railroad. He was also Chief Engineer of the Automobile Highway up Pike's Peak, and held both these positions at the time of his death which occurred at his home in Colorado Springs, Colo., on October 2d, 1915. He is survived by his widow, Mrs. Lillian Branham Gray, and one daughter, Janice.

Mr. Gray was a young man of great promise. Although only 37 years old at the time of his death, he had attained high rank in his profession and had held, for a number of years, positions of such responsibility as seldom fall to so young a man.

He was possessed of a very pleasing personality which won for him friends wherever he went or in whatever task he was engaged, and he held at all times the respect and confidence of those for whom he worked, as well as of those who worked for or under him. Such a combination of capacity, industry, affability, and absolute integrity

* Memoir prepared by the Secretary from information on file at the House of the Society.

is rarely found in one man, and Mr. Gray's death was not only a severe blow to his family and friends, but a great loss to the Engineering Profession to which he was devotedly attached.

Mr. Gray was elected an Associate Member of the American Society of Civil Engineers on May 1st, 1907, and a Member on December 6th, 1910.

LINDSEY LOUIN JEWEL, M. Am. Soc. C. E.*

Died September 5th, 1915.

Lindsey Louin Jewel was born at Christiansburg, Va., on November 24th, 1877, and was graduated in 1900 with the degree of B. S., from the Virginia Polytechnic Institute. In 1902, on the completion of two years' advanced work, the degree of C. E. was conferred on him by the same institution. In 1898, while a student, Mr. Jewel enlisted in the 2d Virginia Volunteers for service in the Spanish-American War, and was mustered out at the close of the War with the rank of Corporal.

Mr. Jewel's first work after graduation was with the Penn Bridge Company, Beaver Falls, Pa., and, while so employed. he designed and built extensions to the sanitary sewer system for New Brighton, Pa. In 1903, he entered the service of the McClintic-Marshall Construction Company, Pittsburgh, Pa., as Designing Engineer, and during the period, 1904 to 1906, designed all the buildings for the Youngstown Sheet and Tube Company, a number of bridges and viaducts on the Virginian Railway, including the New River Bridge, at Glen Lynn, Va., buildings for the Bethlehem Steel Company, etc. In 1906 he was promoted to the position of Manager of Erection, in entire charge of all erection operations west of Harrisburg, Pa., and, while thus engaged, planned and handled the erection of Piers 57, 58, and 59, Chelsea Section, New York City, 14th Street Viaduct, Hoboken, N. J., the Pen Horn Creek Viaduct of the Erie Railroad, New River Bridge of the Virginian Railway, the Ohio River Bridge (cantilever), of the Pittsburgh and Lake Erie Railroad, Harvard Denison Viaduct, Cleveland, Ohio, the Missouri River Bridge, Kansas City, Mo., etc.

In 1910 Mr. Jewel was sent by his Company to the Canal Zone in charge of the lock gate erection for the Panama Canal, and was thus engaged when, in the fall of 1912, he organized the Central American Construction Company, and was chosen its President and Chief Engineer. In the one year of his connection with this company—he returned to the United States in search of health in October, 1913—

* Memoir prepared by James H. Gibboney, Chf. Chemist, Norfolk and Western Railway, Roanoke, Va.

several large docks, the railway station at Panama, and a number of minor structures, were completed.

As a resident of the Canal Zone, Mr. Jewel was actively identified with its civil and social affairs. He was a member of the local clubs, and, in recognition of his interest in development work in the Zone, President Wilson, in 1913, appointed him United States Vice-Consul at Colon.

He died at Saranac Lake, N. Y., on September 5th, 1915, after an illness of nearly two years.

Mr. Jewel was an active member of the National Association of Audubon Societies and the American Ornithological Union, and was the author of a number of papers on Isthmian birds, his rare collection of some 400 specimens attesting to his interest and skill as a collector.

Dr. John M. McBryde, President-Emeritus of the Virginia Polytechnic Institute, adds this beautiful tribute to Mr. Jewel's manly character and scholarship:

"In the opening days of the Session of 1896-97, while President of the Virginia Polytechnic Institute, I was greatly struck by the manly bearing of one of the new matriculates. Indeed, the energy and force of character apparent in every line of his face were such as would favorably impress any intelligent observer. Within a very short time the record made by this young man, Lindsey Louin Jewel, of Christiansburg, Virginia, fully confirmed my first favorable impressions, and before the close of the first term his high standing in his classes and efficient leadership in every healthy form of college activity won for him the approval and esteem alike of professors and students. And from year to year of his collegiate course, I was delighted to see him fully measure up to the high standard of performance exhibited at its outset, and at his graduation with distinction in 1900, I confidently expected for him an exceptionally successful career in active life. Although we met but once or twice after his graduation, I was not disappointed in my expectations, for rising rapidly from year to year in his chosen profession he had, before reaching middle life, won for himself in the short period of time allowed him on earth, a position of wide and commanding influence and placed to his credit a surprising amount of engineering work of the highest grade and value.

"His removal from the scene of his activities at the very time when his promise of abounding usefulness was greatest was a loss not only to his family and section, but to the whole country as well. He left behind him, however, an example of preparation and performance of which his family, his Alma Mater, and his State may well be proud."

Mr. Jewel was elected an Associate Member of the American Society of Civil Engineers on April 4th, 1906, and a Member on November 1st, 1910.

CHARLES CHANCELLOR WENTWORTH, M. Am. Soc. C. E.*

Died November 11th, 1915.

Charles Chancellor Wentworth was a descendant of Colonial Governor Wentworth, of the New Hampshire Colony, and was born in Philadelphia, Pa., on February 21st, 1856. After preliminary schooling at private and public schools and Rugby Academy, he was graduated, in 1876, from the University of Pennsylvania.

In the fall of that year he acted as Levelman on the Seaboard Pipe Line survey from Shippensburg eastward to Baltimore, Philadelphia, and Chester, under Gen. Herman Haupt, Chief Engineer.

In 1877-78, he was employed as Instrumentman on the location and construction of the Foxburg, St. Petersburg and Clarion Railroad, of Pennsylvania, under John Graham, Jr., Chief Engineer.

In 1879, Mr. Wentworth was appointed Assistant Engineer in charge of location of the New River Railroad Company, in Virginia, a line which subsequently became the first outlet of the now well-known Pocahontas coalfield. The construction of this line was carried to completion by the Norfolk and Western Railroad, and, in 1881, Mr. Wentworth was employed on this work under the Chief Engineer of the latter road.

From 1883 to 1889, he was Assistant Engineer to the Chief Engineer of the Norfolk and Western Railroad, with his main office in Roanoke, Va. During that period he was in direct charge of the various engineering construction works, including tidewater and division terminals, branch lines, shops, stations, much bridge work, and other forms of railroad construction, including the improvements in connection with the development of the Pocahontas coalfield.

In 1889, Mr. Wentworth left the service of the Norfolk and Western Railroad to become Chief Engineer of the American Bridge and Iron Company, of Roanoke, Va. While in this position he designed and put into effect a system of standardized highway bridge construction whereby bridges of 200 ft. span or less, including drawbridges, could be constructed, through uniformity of design, with a minimum amount of office work. This system has been used for many years by that Bridge Company and its successor Company.

In 1898, Mr. Wentworth left the American Bridge and Iron Company and re-entered the service of the Norfolk and Western Railway Company as Bridge Engineer. During a number of years he made a study of velocity grades and their effect in the revision of existing track, as governed by the tractive power of a locomotive at varying speeds.†

* Memoir prepared by Charles S. Churchill, M. Am. Soc. C. E.

† The results of these studies were published in the *Railroad Gazette*, December 8th, 1899, *et seq.*

He also made a study of the theory of railroad curves, with a view to establishing positive and practical methods of assuring the easy motion of cars.*

In 1901 he made an analysis of the theory and design of hydraulic rams, in order to extend their application to pumping large quantities of water. For this he was awarded the Longstreth Medal by the Franklin Institute on December 3d, 1902.

In the same year, in collaboration with the writer, he made a considerable detailed study of the delivery of fresh air through tunnels, the first application of which was made to Elkhorn Tunnel, on the Norfolk and Western Railway, which method afterward became known as the Churchill-Wentworth System of Tunnel Ventilation.†

On February 1st, 1903, Mr. Wentworth was appointed Principal Assistant Engineer of the Norfolk and Western Railway, which position he held until his death.

He was a very active member of the American Railway Engineering Association, and served on various committees thereof. He was in generally good and vigorous health until suddenly overtaken by death while asleep. He is survived by his widow, Sophia Park Wentworth, three sons, and three daughters.

Mr. Wentworth was elected a Member of the American Society of Civil Engineers on April 4th, 1888.

THOMAS HOVENDEN, Jr., Assoc. M. Am. Soc. C. E.‡

Died September 19th, 1915.

Thomas Hovenden, Jr., son of Thomas Hovenden, the well-known artist, and Helen C. Hovenden, was born at Plymouth Meeting, Pa., on March 11th, 1882. His early education was received at the Friends' School, at Plymouth Meeting, and the Friends' Select School, in Washington, D. C., from which he was graduated in 1899. He then entered the Civil Engineering Department of the University of Pennsylvania, and was graduated in 1903 with the degree of B. S.; and received that of M. S. in 1904.

For a year after his graduation Mr. Hovenden held the position of Instructor in the Civil Engineering Department of the University of Pennsylvania, leaving, in 1905, to enter the employ of W. W. Lindsay and Company, Engineers and Contractors, of Philadelphia.

* "Line and Surface for Railway Curves", *Transactions*, Am. Soc. C. E., Vol. XLVIII, p. 357.

† The formulas have been given by Mr. Wentworth in *Transactions*, Am. Soc. C. E., Vol. LIV, Part C, p. 567.

‡ Memoir prepared by W. W. Lindsay & Co., Harrison Bldg., Philadelphia, Pa.

From that time until his death, he remained with this firm (whose business is steel-plant and mill-building construction), and rose from a subordinate position to that of General Superintendent, in charge of all construction work done by the Company.

During this time Mr. Hovenden had charge of a great variety of work in steel, concrete, and timber construction. He built several open-hearth and heating furnaces and manufacturing plants of various kinds in the Eastern States. For some months he was in Cuba in charge of the erection of a number of highway bridges for the Cuban Government, for which his firm had the contract.

His sterling character and his ability, which was of the highest order, were much appreciated by his employers and by all with whom he came in contact, and his untimely death, which occurred on September 19th, 1915, at the age of thirty-three, cut short a career of much promise.

Mr. Hovenden was elected an Associate Member of the American Society of Civil Engineers, on July 9th, 1912.

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"COHESION IN EARTH: THE NEED FOR COMPREHENSIVE EXPERIMENTATION TO DETERMINE THE COEFFICIENTS OF COHESION." WILLIAM CAIN. (To be presented Feb. 2d, 1916.)

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PROGRESS REPORT OF THE SPECIAL COMMITTEE ON A NATIONAL WATER LAW. (To be presented Jan. 19th, 1916.)

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